VULNERABILITY OF LOW COST MASONRY BUILDINGS TO EARTHQUAKES

ABSTRACT

Thesis submitted for the degree of

Doctor of Philosophy

in

CIVIL ENGINEERING

By

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ALIGARH MUSLIM UNIVERSITY
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August, 2006

ABSTRACT

There has been a need for low cost housing in India and many other parts of the world for decades. The cost of tradition building material has gone up considerably over the past few years due to inflation, increasing cost of energy and also due to the widening gap between demand and supply because of rapid population growth. Consequently, the cost of housing has increased in recent years posing great challenges to the engineers. By the use of traditional material and construction methods alone, it will not be possible to construct houses at a pace matching the ever increasing demand. Even though considerable R&D work has gone into development of a large number of economical and efficient building materials and low cost construction techniques, their application in the housing sector has not attained the desired level. There is an urgent need for reducing the time and cost of construction by adopting improved production techniques, rationalizing design methods for efficient use of even the traditional materials, by using innovative techniques and material for housing construction. There is also a need to educate the public that low cost housing does not necessarily mean inferior housing. What is really required at the moment is to win the confidence of the people, for wider acceptance of such low cost dwellings. This is only possible if people are insured that these houses are economical, comfortable, fulfill the requirement of a good housing and are safe during earthquakes.

Fired earth brick is the most popular low cost building material used in the third world countries. Some innovative techniques have been recently proposed for making some saving in the cost of construction of brick masonry walls. These construction techniques involve the construction of brick masonry cavity walls and 190 *mm* thick reduced thickness wall by using the conventional solid bricks (228×114×76 *mm*). Though considerable saving in the cost can be achieved by such techniques but its adoption in practice requires thorough understanding about their mechanical properties and the

response of such structures to lateral loads especially the earthquake forces. The present research programme is aimed at attaining these objectives so as to provide a rational design basis for such low cost buildings.

Four types of brick masonry viz. Type A, B, C and D have been used in the present study. The brick masonry Type A and B are solid, whereas Type C and D are having inside cavity. In brick masonry Type A, bricks in every course are flat, in Type B one brick is flat and one on-edge in every course. In masonry Type C, there are alternate header and stretcher courses, header course is of flat bricks and stretcher course is of on-edge bricks. In masonry Type D, bricks in every course are on-edge with alternate header and stretcher bricks.

The advantages of such types of low cost brick masonry (Type B, C and D) options over the conventional solid brick masonry (Type A) are:

i) Use of 228 *mm* cavity wall and 190 *mm* thick wall results in the saving of bricks by 25% & 16% respectively.
ii) Reduction in the number of bricks used will result in the reduction of dead load of the superstructure, thereby reducing the cost of foundation.
iii) Wall surface becomes smooth and gives better appearance, hence no plastering is required.
iv) Reduced thickness of 190 *mm* thick wall as well as 228 *mm* cavity wall also reduces the quantity of cement mortar to be used for the construction of such walls.
v) Opening inside the cavity walls act, as an insulator.
vi) Due to the opening inside the cavity wall electrification can be done easily.

The following studies were carried out to achieve the desired goal:

1. Basic mechanical properties of different types of brick masonry such as compressive, shear and bond strength

2. Behaviour of wall panels constructed with different types of brick masonry under lateral static load
3. Performance of building models constructed with different types of brick masonry under lateral static load
4. Assessment of lateral stiffness of wall panels and shear walls of building models
5. Performance of building models with and without base isolation constructed with different types of brick masonry under dynamic load

Extensive experimental testing of brick masonry prisms, wall panels and building models was carried out. Seismic resistance of building models was also tested with and without base isolation using shake table facility. Detailed analysis of these masonry buildings were carried out for simulating their seismic response.

In most of the brick masonry prisms tested under compression, cracks developed along the vertical mortar joints. All the masonry prisms tested in shear failed by sliding along the horizontal mortar bed which is the weakest plane in shear. In the case of wall panels tested under static lateral load; wall panels of masonry Type A and B failed due to sliding at the base along the horizontal mortar bed which is because of the masonry being stronger along the diagonals. Whereas, the wall panel of masonry Type D failed in diagonal tension by the development of cracks along the diagonal passing through horizontal and vertical mortar joints because this type of masonry is relatively weaker in tension along its diagonals. In the building models tested under static lateral load, cracks developed are mainly diagonal, whereas, some of the cracks are horizontal and close to the slab. All the cracks observed in different building models are along the mortar joints.

The compressive strength of masonry Type D is maximum whereas, shear strength is slightly less than masonry Type B. The inclusion of shear deformation and structural opening considerably reduces the lateral stiffness of wall.

This study also confirmed that the performance of building models with base isolation is much better as compared to building models with out base isolation. Thus the sliding arrangement shows great promise for adoption in actual building construction as a measure of earthquake safety.

On the basis of the mechanical properties, behaviour of wall panels and the performance of building models under static as well as dynamic loads, the masonry Type D is found to be better than masonry Type B. Though the saving in the quantity of bricks and mortar as well as compressive strength is maximum in masonry Type D, but the shear strength is slightly less than that of the masonry Type B. The introduction of base isolation by providing Teflon sheet at the plinth level of the building improves its performance under earthquake forces.

Thus we can say that the present study have provided a good basis for the extension of low cost construction practices in the seismically active areas. People's confidence in their seismic performance will give rise to the construction of such low cost dwellings on commercial scale.

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T6403

Dedicated to My Parents
As a Token of Love and
Affection

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ALIGARH (U.P.) INDIA

CERTIFICATE

Certified that work embodied in this thesis entitled, " **VULNERABILITY OF LOW COST MASONRY BUILDINGS TO EARTHQUAKES** " is the result of original researches carried out by **Mr. Iqbal Khaleel Khan** under my supervision and is suitable for submission for the award of **Ph. D.** degree of Aligarh Muslim University, Aligarh.

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ACKNOWLEDGEMENT

I bow down before the Almighty who is most merciful and benevolent to me due to whom I am in a position to submit this thesis for the award of the Ph. D. degree of Aligarh Muslim University, Aligarh.

It gives me great pleasure to take this opportunity of acknowledging my deep sense of gratitude and obligation to my supervisor Dr. Husain Abbas, Professor, Department of Civil Engineering, Aligarh Muslim University, Aligarh for his able guidance and constant encouragement, without which this work could not have been completed.

I gratefully acknowledge the financial support given by All India Council for Technical Education (AICTE), New Delhi, India for carrying out the present study.

I am very much thankful to Prof. M. Jamil, Chairman, Department of Civil Engineering, Aligarh Muslim University, Aligarh for providing his affectionate support, encouragement and facilities in the Department.

I am really indebted to all my colleagues and friends for their fruitful discussions and encouragement from time to time during this work.

It is indeed my privilege to put on record my deep sense of love and gratitude to my loving daughters Ramsha, Tooba and Juveria who suffered a lot, while I was busy in the completion of this work.

I shall be failing in my duties if I do not record the help, constant encouragement, inspiration and support that I received from my wife Dr. Farah Sajjad.

7.8.6

(Iqbal Khaleel Khan)

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NOMENCLATURE

L	length of brick
B	width of brick
D	thickness of brick
B_1	least lateral dimension of masonry prism
B_2	other lateral dimension of masonry prism
C_f	correction factor
h	height of masonry prism
r	minimum radius of gyration
$\frac{h}{r}$	slenderness ratio
r'	sum of ratio of mortar mix
τ	shear strength of brick masonry
σ	compressive stress
σ_b	compressive strength of brick
σ_m	compressive strength of mortar
σ_{bm}	compressive strength of brick masonry
ε	compressive strain
a, b	model parameters
α, β	constants
K	coefficient which depends on layout of bricks and joints
K_s	spring constant
K_c	coefficient for compressive strength of brick masonry
K_{ss}	coefficient for shear strength of brick masonry
K_r	coefficient for radius of gyration
K_1, K_2	coefficients which depend on the type of mortar

s_b	standard deviation for the strength of brick
s_m	standard deviation for the strength of mortar
s_{bm}	standard deviation for the strength of brick masonry
δ_b	coefficient of variation for the strength of brick
δ_m	coefficient of variation for the strength of mortar
δ_{bm}	coefficient of variation for the strength of brick masonry
δ_s	shape factor to account for the shape and size of brick
δ_{mf}	moisture factor to account for the moisture content of brick masonry
P	horizontal force applied at the top of wall to give unit lateral displacement
U	strain energy
M_x	bending moment at a section distant x from base
M	fixed end moment
F	shear force
V	support reaction
q	shear stress intensity at a section
z	width of the fibre at a distance y from neutral axis
y	distance of fibre under consideration from neutral axis
y_t	distance of the fibre in tension zone under consideration from neutral axis
y_c	distance of the fibre in compression zone under consideration from neutral axis
$A\bar{y}$	moment of area of the portion which is between the fibre under consideration and the extreme of fibre
L_w	length of wall
H	height of wall
t_w	thickness of wall
L_1	width of door
H_1	height of door
L_2	width of ventilator
H_2	height of ventilator
L_3	width of window

H_3	height of window
I	second moment of area of wall about neutral axis
I_1	second moment of area of wall for the portion having door opening about neutral axis
I_2	second moment of area of wall for the portion having ventilator opening about neutral axis
I_3	second moment of area of wall for the solid portion without opening about neutral axis
I_4	second moment of area of wall for the portion having window opening about neutral axis
E	modulus of elasticity of brick masonry
G	shear modulus of brick masonry
M_t	mass lumped at roof level
M_b	mass lumped at base
M_T	sum of top and bottom mass i.e. total mass
$\ddot{X}_t$, $\ddot{Z}_t$	absolute and relative accelerations of the top mass respectively
$\ddot{X}_b$, $\ddot{Z}_b$	absolute and relative accelerations of bottom mass respectively
$\ddot{y}(t)$	ground acceleration at any instant of time, t
Z_b, Z_t	lateral relative displacements of bottom and top masses respectively
$\dot{Z}_t$, $\dot{Z}_b$	relative velocities of bottom and top masses respectively
S_f	force to cause sliding
g	acceleration due to gravity
C_s	coefficient of viscous damper
ω	natural circular frequency of the system
ξ	fraction of critical damping
μ	coefficient of friction
θ	mass ratio $= \dfrac{M_t}{M_b}$

Chapter 1
INTRODUCTION

1.1. GENERAL

Fired earth brick masonry structures are used by human being since ancient days and even today brick is one of the most common and popular low cost construction material in the third world countries. But due to the rapid population growth, providing shelter to every one is becoming difficult. The problem is also getting aggravated because of escalation in the cost of construction materials. Therefore, there is an urgent need that some innovative techniques may be developed so that the cost of the structures can be considerably reduced and the dwellings become affordable for common man.

Along with the reduction in the cost of structures, it is also important that such low cost houses become popular among the masses irrespective of the geological conditions of the area where these are located. This will be possible when along with the reduction in the cost of the structures, their safety against natural calamities like severe wind force and earthquake force is ensured.

Therefore a house should be affordable and at the same it should be safe against earthquake forces. Unfortunately masonry structures are constructed without testing their performance under earthquake forces. By making suitable provisions during the construction of brick masonry structures, amount of destruction, loss of life and money can be considerably reduced.

Though a number of cost effective solutions for brick masonry construction have been proposed in the recent past but most of them could not become popular because these are not supported by rational design procedure. Many earthquakes in the past have witnessed the collapse of a large number of such low cost dwellings resulting in innumerable fatalities.

Some of the prevalent low cost brick masonry construction techniques involve the construction of brick masonry cavity walls (one brick thick i.e. thickness of wall = L) and reduced thickness walls (thickness of wall = $B+H$) by using the conventional solid bricks of size $L\times B\times H$ where L is its length, B is the width and H is the thickness. Though considerable saving in the cost can be achieved by such techniques but their adoption in practice requires thorough understanding about their mechanical properties and the response of such structures to lateral loads especially the earthquakes. The present study has been conducted for assessing the performance of such low cost buildings and to improve their performance for resisting the earthquake forces.

1.2. BRICKS AND BRICK MASONRY

All over the world bricks are available in large variation of shapes and sizes, from approximately 120×90×45 *mm* to 600×600×600 *mm*. The actual sizes of the bricks are the nominal sizes minus the joint thickness. Clay bricks can be solid or hollow. In the U.S.A. they are defined as solid if the net cross-sectional area in every plane parallel to the bearing surface is 75% or more of its gross cross-sectional area measured in the same plane. They are defined as hollow, if the cores, cells or hollow spaces within the total cross-sectional area exceed 25% of the cross section of the unit. The cores can have different shapes, and over 100 cores in a brick have been used. The density of solid clay bricks ranges from 13 to 22 kN/m^3. The dimensions of some of the common types of bricks are given in Table 1.1.

The size of traditional bricks used varies from 210 to 250 *mm* in length, 100 to 130 *mm* in width and 60 to 75 *mm* in thickness in Indian subcontinent. The commonly adopted nominal size of a traditional brick in Indian Subcontinent is 228×114×76 *mm*. This size of brick has been decided such that it could be easily burnt to the core and its weight should be such that the mason could conveniently lift and place it with one hand without fatigue. In general, the length of brick is kept twice its width plus the thickness of one mortar joint. The Indian bricks have got a frog whose size vary i.e. from 150 to 170 *mm* in length, 40 to 60 *mm* in width and 8 to 12 *mm* in depth. The shape of frog is usually rectangular in plan with rounded corners.

Table 1.1 Dimensions of Different Types of Bricks

Type of Bricks	Length, *L* (*mm*)	Width, *B* (*mm*)	Thickness, *H* (*mm*)
Indian Brick	203	101	67
Roman Brick	304	101	50
Norman Brick	304	101	67
Engineer's Brick	203	101	81
Economy Brick	203	101	101
Jumbo Brick	304	101	101
Double Brick	203.2	101	135
Triple Brick	304	101	135
Indian Brick	228	114	76

Normally we use 1:6 cement and sand mortar for the brick masonry. Before laying the bricks, these are soaked in water for atleast 6 to12 *hrs*. The bricks are laid such that their frogs are upward and filled with mortar. The vertical joints are properly filled by applying mortar on the sides of bricks. After the construction brick masonry, it should be cured for atleast 7 to 14 *days*.

1.3. COST EFFECTIVE BRICK MASONRY

A number of cost effective measures are being adopted in the construction of brick masonry buildings such as the construction of brick masonry cavity walls and solid walls of reduced thickness. Such buildings have been constructed mainly in Southern India by

British Architect Mr. Laury Becker who has propagated the idea. The brick masonry cavity walls and solid walls of reduced thickness are built using conventional solid fired earthen bricks. The saving in the cost and merits of such type of construction are:

i) Use of one brick thick cavity wall and solid wall of reduced thickness results in the saving of bricks by 25% and 16% respectively.

ii) Reduction in the number of bricks reduces the quantity of mortar and results in the reduction of dead load of the superstructure, thus reducing the cost of foundation.

iii) Wall surface becomes smooth and gives better appearance, hence no plastering is required.

iv) The cost of brick masonry walls is about 35% and that of the plastering is about 10% of the total cost of building, thus the saving in the cost of building due to the saving in the number of bricks, masonry mortar and plastering used as well as due to reduced cost of foundation is about 20-25%. Though there will be some increase in the cost of labour because of the specialized construction but the mason has to handle less number of bricks, therefore, the net increase in the cost of labour is almost nil which has also been observed in practice.

v) Cavities in one brick thick cavity walls act as an insulator and reduce the chiseling work done for electrification purposes.

1.4. EARTHQUAKE DAMAGES

In the recent past many earthquakes in our country have witnessed a lot of destruction mainly because of the ignorance of earthquake forces at the time of design and construction of structure in seismically active zones. The existing states of art can be better understood by the following facts.

On August 15, 1950, 1,538 people were killed in earthquake measuring 8.5 on Richter scale in Assam. About 1,000 people were killed in Bihar earthquake on August 20, 1988.

An earthquake measuring 6.6 on the Richter scale occurred in Uttarkashi on October 20, 1991 and killed 1,500, persons.

On September 30, 1993 at 00:03:53 hours (Indian Standard Time), a killer earthquake of 6.4 magnitude flattened 52 villages in the two districts of Latur and Osmanabad, situated in Maharashtra, India. The epicenter was located near the village of Killari, Latur district, Maharashtra State, Central India (18.2^0 N, 76.4^0 E) at a depth of about 5 *km*. Widespread death and destruction in the districts of Latur and Osmanabad, Maharashtra state; complete destruction of stone /mud structures in about 20 villages covering an area of about 15 *km* wide and centered 5 *km* west of Killari. The death toll was about 10,000 and left 16,000 injured. A village Killari with a population of over 20,000 was completely destroyed and over 1,400 persons killed. As a whole, about 1,87,000 houses were either completely destroyed or damaged to irreparable extent and loss of property worth Rs. 12 *billion* (= US$ 0.24 *billion*) was estimated. To rehabilitate the affected persons, government spent about Rs. 18 *billion* (= US$ 0.36 *billion*), mostly on dwellings. The earthquake occurred in the middle of the night when most people were indoors and therefore particularly vulnerable. Engineered structures were relatively scarce in the affected area. A maximum intensity level of MMSK VII-IX could be determined by the performance of the few constructed brick-and-mortar structures. The collapse of traditional stone-and-mud buildings in the mesoseismal area was nearly total.

The "Jabalpur Earthquake" measuring 6 on the Richter scale occurred on May 22, 1997 at 04:22 am (Indian Standard Time) centered about 8 *km* southeast of the city of Jabalpur (23.18^0, N 80.02^0 E) in the state of Madhya Pradesh in central India. It caused significant damage to structures in the districts of Jabalpur, Mandla, Sivni and Chhindwada in the state of Madhya Pradesh. The maximum damage was in the districts of Jabalpur and Mandla. About 8,546 houses collapsed and about 52,690 houses were badly damaged. During this earthquake, about 38 persons died and about 350 were injured. The affected area lies in peninsular India and is along the Precambrian Narmada rift zone. The maximum intensity of shaking experienced during the earthquake was VIII on the MMSK scale.

An earthquake of magnitude 6.6 occurred close to the India - Nepal border on August 21, 1998 at 4:39:11 hours (Indian Standard Time). The epicenter was located in eastern Nepal between Udaipur and Dharan (26.7° N, 86.8° E). The focal depth was estimated to be about 36 miles. Widespread devastation and loss of life was reported. One thousand and four people died (282 in India and 722 in Nepal) and more than 16000 were injured. The affected area was mainly in the Gangetic alluvial plain of Bihar (India) and Nepal, and the hilly regions of eastern Himalayan ranges. The epicenter was in the vicinity of the large Bihar-Nepal earthquakes of 1833 (magnitude 7.0-7.5) and 1934 (magnitude 8.4). There was significant damage to the embankments, railway bridges and buildings in Bihar. In addition, hilly regions of Darjeeling district (in the state of West Bengal) and Sikkim, located far away (approximately 125 *miles*) from the epicenter, sustained extensive damage, including damage in roads and highway bridges. Through this earthquake, nature conducted a real-life full-scale test on construction practices in India as well as on our post-earthquake performance and ability to respond to earthquakes.

The Chamoli (Himalaya, India) earthquake of 29 March, 1999 in northern India is yet another important event from the viewpoint of Himalayan seismotectonics and seismic resistance of non-engineered constructions. The earthquake caused death of about 100 persons and injured hundreds more. Maximum MMSK intensity was up to VIII at a few locations. Most of the damage was that of low cost masonry or stone houses.

On January 26, 2001, when the entire country had readied to celebrate 52nd Republic day, an earthquake measuring 7.9 on Richter scale with its epicenter 20 *km* north-east of Bhuj in Kutch district of Gujarat at 8:45 *am* (IST) on Friday again shook the ground and left 18,600 persons dead and over 167,000 injured.. The destruction caused by this earthquake was so intense that it is hard to say that this town ever existed on the surface of earth. Almost the entire town, the biggest in the Kutch, had been razed to the ground and the death toll here alone was put at 13,000. The estimated economic loss due to this earthquake is placed at around Rs. 220 *billion* (= US$ 4.40 *billion*). Besides, low cost houses, numerous multi-storey RC frame buildings also collapsed even in distant towns.

These are the few examples of earthquakes that occurred in India in the recent past and we have also seen the pace of destruction and loss of life caused by these earthquakes. A summary of earthquakes that occurred in India is given in Table 1.2. Some of the severe destructions caused to buildings due to killer earthquakes have been shown in Fig. 1.1. Most of the damages during the past earthquakes in India have been caused by the collapse of low cost houses which are constructed without any rational basis. Some of the factors that contribute to the poor performance of the traditional structures are unavoidable such as the use of only the available materials and the economic constraints. Some patterns of failure, however, point to improvements that probably could be made within these limitations.

Fig. 1.1 Damage of Low Cost Buildings Located in Bachau Town Caused by Gujrat Earthquake (2001)

In view of the above facts it can be said that while minimizing the cost of structure using innovative techniques during construction, attention should also be paid towards the safety of the structure during natural calamities like earthquakes, windstorm and hailstorm etc.

The timing of the earthquake during the day and during the year critically determines the number of casualties. Casualties are expected to be high for earthquakes that strike during cold winter nights, when most of the population is indoors.

Table 1.2 Major Past Earthquakes in India

Date	Place	Time (IST)	Magnitude (Richter Scale)	Maximum Intensity (MMSK Scale)	Deaths
16 June, 1819	Cutch	11:00	8.3	IX	1,500
12 June, 1897	Assam	16:25	8.7	XII	1,500
08 Feb., 1900	Coimbatore	03:11	6.0	VII	Nil
04 Apr., 1905	Kangra	06:10	8.0	X	19,000
15 Jan., 1934	Bihar	14:13	8.3	X	11,000
15 Aug., 1950	Assam	19:39	8.6	X	1,530
21 July, 1956	Anjar	21:02	6.1	IX	115
10, Dec., 1967	Koyna	04:30	6.5	VIII	200
23 March, 1970	Bharuch	20:56	5.2	VII	30
21 Aug., 1988	Bihar	04:39	6.6	IX	1,004
20 Oct., 1991	Uttarkashi	02:53	6.4	IX	768
30 Sep., 1993	Killari	03:53	6.2	VIII	7,928
22 May, 1997	Jabalpur	04:22	6.0	VIII	38
29 March, 1999	Chamoli	00:35	6.6	VIII	63
26 Jan., 2001	Bhuj	08:46	7.7	X	13,805

1.5. OBJECTIVES AND SCOPE OF WORK

For making the housing affordable for the common man, the construction of low cost masonry buildings has recently been started but surprisingly without any basic research. In the absence of such study, either the reserve strength of such masonry will remain unutilized or may not be capable of resisting the forces to which these buildings may be subjected. Instead of waiting to observe their performance under severe natural loads like earthquake and cyclones and to be a mute spectator to self made disaster, there is an urgent need to study their performance in laboratory. It is with this objective that the present study was taken up. The response of load bearing masonry buildings depend upon the basic properties of brick masonry with which it has been constructed, therefore, there is a need to establish these properties for such low cost masonry construction.

Though such constructions may be good enough for vertical loads but their resistance to lateral loads requires supporting experimental evidence. In view of the devastating earthquakes witnessed in the recent past in our country, there is an urgent need of evaluating the seismic resistance of such low cost dwellings before their implementation in the seismically active zones.

The study presented in the thesis focuses on load bearing brick masonry structures such as one brick thick brick masonry cavity walls and solid wall of reduced thickness using conventional solid bricks. The types of low cost masonry that have been considered in the present study are: (i) conventional brick masonry in English bond referred in the thesis as Type A; (ii) solid wall of reduced thickness masonry in English bond termed in the thesis as Type B; and (iii & iv) Rat-Trap brick masonry in English and Flemish bond henceforth referred to as Types C and D. For establishing a relationship between the model and prototype, tests have been performed on Type A brick masonry built with full size bricks (228×114×76 *mm*) having frog of size 170×60×8 *mm* and model brick of half size (114×57×38 *mm*) having frog of size 85×30×4 *mm*. This relationship between model and prototype for Type A masonry has been assumed to extend to other types of masonry also.

The brick masonry Type A and B are solid, whereas Types C and D are hollow. The English bond in brick masonry has alternate courses of header and stretcher, whereas, Flemish bond has alternate header and stretcher bricks in every course. In brick masonry Type A, bricks in every course are flat; in Type B one brick is flat and one on-edge in every course. In masonry Type C, there are alternate header and stretcher courses, header course is of flat bricks and stretcher course is of on-edge bricks. In masonry Type D, bricks in every course are on-edge bricks. All the four types of brick masonry can be seen in Fig. 1.2.

The study will provide a good basis for the extension of low cost construction practices for seismically active areas. People's confidence in their seismic performance will give rise to the construction of such low cost dwellings on commercial scale.

The research plan is proposed to cover the following:

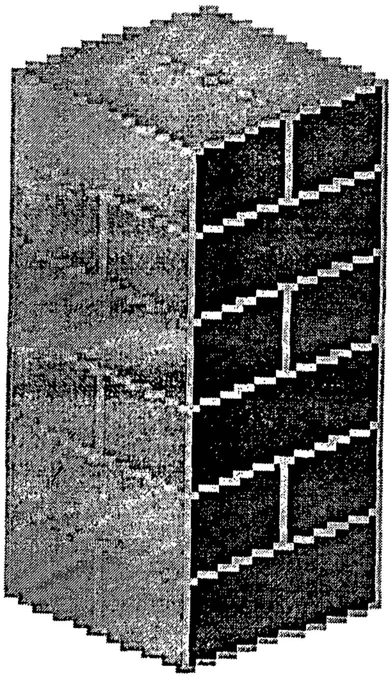
i) Determination of basic mechanical properties of low cost brick masonry such as compressive and shear strength by testing masonry prisms and wall panels.

ii) Experimental determination of seismic resistance of buildings made with such walls by scaled model testing of buildings.

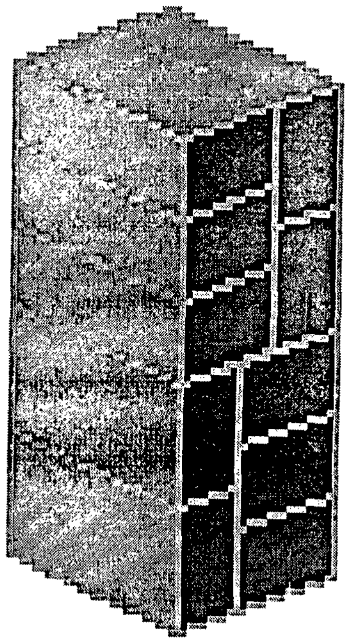
iii) Testing of models of low cost single storey masonry buildings with base isolation for assessing the performance.

iv) To develop mathematical models for the seismic analysis of such low cost buildings and to validate the results with experiments.

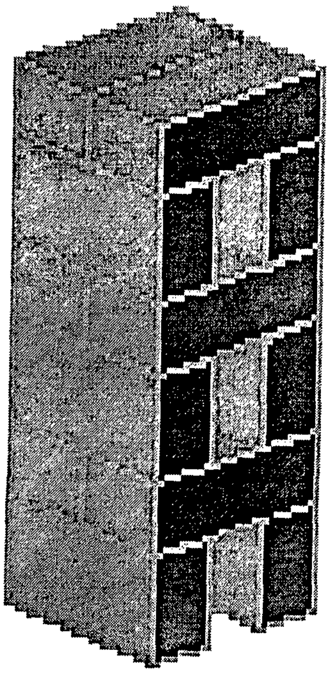
v) To develop a rational procedure for design and construction of such buildings by employing the mechanical properties determined for this purpose and to suggest base isolation technique suitable for such buildings.



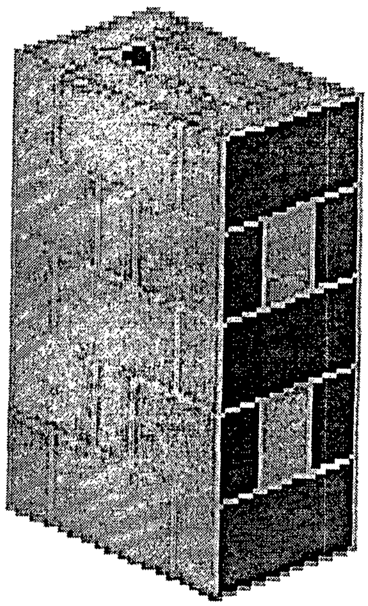
(a) Type A Brick Masonry



(b) Type B Brick Masonry



(c) Type C Brick Masonry



(d) Type D Brick Masonry

Fig. 1.2 Different Types of Brick Masonry

1.6. LAYOUT OF THESIS

Chapter 1, the present chapter deals with the historical background in mastery of materials and construction methods. The historical developments in the area of cost optimization of residential buildings are given in this chapter. The objectives and scope of the work are presented in this chapter.

The literature in the areas relevant to the field has been critically reviewed in the second Chapter. This chapter has details about alternate materials, alternate methods of construction with conventional material and policy and programme with respect to shelter development for low cost housing. The prevalent practices for reducing the cost of structure are discussed.

The experimental work carried out in the present study has been reported in the third Chapter. It has details about the materials used, the making and testing of masonry prisms, wall panels and building models under static as well as dynamic loads. The properties of the materials used are given.

The fourth Chapter contains analysis of experimental data and discussion of results observed in the present study. Mathematical model for seismic analysis of low cost masonry buildings is presented in this Chapter. The lateral stiffness of masonry walls and building models built with different types of masonry and with different positions of opening has been evaluated. The finite element analysis of masonry wall for varying thickness of joints is also given in this Chapter.

Conclusions derived from the experimental work and analysis of results, have been discussed in the fifth Chapter. Suggestions have been made for future extension of the present study.

Chapter 2

LITERATURE REVIEW

2.1. INTRODUCTION

Brick masonry is being used in the construction industry from the time immemorial for the construction of walls and columns to resist compressive loads. Factors which affect the compressive strength of brick masonry include brick strength, mortar strength and joint thickness. Most of the research work carried out in this area, either for the prediction of strength of brick masonry or to study the performance of brick masonry buildings is for the conventional brick masonry. There is very little literature available regarding the strength prediction and performance study of low cost brick masonry. The available published literatures that are relevant to the present study have been exhaustively reviewed and presented in this Chapter. It does provide an overview of the research carried out in the field of conventional brick masonry, which will also be useful in understanding the behaviour of low cost brick masonry.

2.2. COST OPTIMIZATION OF BRICK MASONRY

Housing must be examined not only in terms of providing shelter but as a means of social policy to achieve growth with social justice. From the social as well as economic point of view it could be very useful to involve the future owners of the house in the construction of dwellings. This would in fact open new possibilities of employment for the presently unemployed and also help to train semi-skilled and skilled people who can in fact be employed on future building sites. In fact it would be production by the masses to some extent rather than production for them.

Before going for the construction of a house, one should ask a simple question, why and why not? Why things are being followed blindly in the construction industry and why could cost not be reduced by following a rational approach and by not being wasteful. At every step one should ask himself, is this necessary?

The terminology "Low-Cost Housing" is sometimes misleading, since it does not make a reference for comparison. It can be associated with almost any reasonable cost. However, "Lower-Cost Housing" carries a clear meaning. There are different techniques employed in India for reducing the cost of dwellings. The cost of dwellings can be reduced by reducing the cost of different components of buildings such as foundation, wall, flooring, slab, door and window etc. by the use of alternate materials or by using the alternate methods with the conventional materials. The goal is being the reduction of existing cost.

2.2.1. Alternate Materials

Advances in science and technology have made it possible to shift our dependence on alternate and cost effective construction materials. Heavy industries generate variety of solid wastes and by-product such as fly ash, phosphor-gypsum, slag, red mud, lime sludge etc. that can be used as alternate construction materials.

Mankind has been familiar with several applications of composite building materials for their housing and building needs. Most of these were based on timber, bamboo, jute and a large variety of vegetable fibres, such as reinforced mud blocks for walls, panels for partition and roofing.

2.2.2. Alternate Methods with Conventional Materials

Brick masonry with cavity and reduced thickness brick masonry constructions are alternate methods with conventional materials that considerably reduce the cost of masonry structures because it results in the saving of bricks and mortar used.

Use of 228 *mm* thick cavity wall with solid bricks as an alternate of 228 *mm* thick solid wall reduces about 25% of the total number of bricks and 40% of the mortar used. This is not only economical but also strong, aesthetically pleasing and has better insulation properties.

Use of reduced thickness (190 *mm* thick) wall as an alternate of 228 *mm* thick solid wall is resulting in the saving of bricks and mortar by 16% and 30% respectively.

2.3. MECHANICAL PROPERTIES OF BRICK MASONRY

To study the behaviour of various brick masonry options used for different house models, it is essential to know the strength of various types of masonry used in the construction. As revealed by various investigators [32, 71, 83], strength of brick masonry depends upon many factors: (i) strength of brick; (ii) strength of mortar; (iii) strength of bond; and (iv) method of bonding of bricks etc.

Deodhar and Patel [26] investigated that as strength of brick increases, or as richness of cement mortar increases, strength of brick masonry also increases. Though, the effect of cement content in mortar on compressive strength is not significant, but it improves the strength of brick masonry to some extent. They developed a mathematical model to ascertain compressive strength of brick masonry if that of brick and cement-sand mortar ratio, i.e., cement content in mortar are known. Based on their studies they have suggested following equation to get the compressive strength of brick masonry prisms; σ_{bm} (*MPa*):

$$\sigma_{bm} = \sigma_b - 0.5r' \quad \text{...(2.1)}$$

where, σ_b is the compressive strength of bricks in *MPa* and r' is the sum of ratio of mortar mix. However this model holds good for brick strength ranging from 5.67 to 13.68 *MPa* and mortar with cement-sand ratio varying from 1:3 to 1:8 and hence can not be generalized. Further, this model is in the form of straight line which can not be true in all the cases.

Deodhar [27] investigated the strength of brick masonry prisms in compression, carrying out experimental work in three stages. In the first stage bricks from various parts of India were tested in compression and grouped in eight classes based on their compressive strength varying from 4.16 to 15.23 *MPa*. Brick masonry prisms were then cast and tested using mortar with different cement-sand ratios to determine the crushing strength. The crushing strength of these prisms for all the classes of bricks and mortar with different cement-sand ratios was investigated. The size of brick masonry prism was varying from 210×210×210 *mm* to 240×240×240 *mm* depending upon the size of bricks but in all cases, aspect ratio was maintained to be approximately one. Mortar with different cement-sand ratios used were 1:3, 1:4, 1:5, 1:6 and 1:8 and joint thickness was maintained as 10 *mm* in all the cases. The fineness modulus of sand used for mortar was 2.01.

In the second stage three types of bricks i.e. metric bricks of size 190×90×90 *mm*, conventional bricks 220×100×70 *mm* and thin bricks of size 220×100×30 *mm* were manufactured from locally available black cotton soil and tested in compression. Brick prisms of size 190×190 *mm* to 220×220 *mm* in plan and height varying from 185 *mm* for metric bricks with 5 *mm* joint thickness to 240 *mm* for conventional bricks with 15 *mm* joint thickness were cast, cured for 28 *days* (by wet gunny bags) and tested in compression. Mortar with cement-sand ratios as 1:4 and 1:6 was used to bond the brickwork. To bond the brickwork, three mortar thickness i.e. 5, 10 and 15 *mm* were used. However, vertical joint thickness between two bricks was maintained to be 10 to 20 *mm* only to have square plan area. The varying joint thickness between the courses was the reason for variation in the height of the brick prism. The ratio of height of brick prism to its lateral dimension was found to be varying from 0.947 for metric bricks to a maximum value of 1.09 for conventional bricks. This was done mainly to find the effect of total thickness of mortar and that of bricks on compressive properties of brick masonry prisms. The compressive strength of these three types of prisms was found as 6.43, 5.80 and 5.30 *MPa*, respectively.

In the third stage, brick masonry prisms of size 200×200 *mm* (in plan) and 400 *mm* in height were cast to plot stress-strain curve for brick masonry. The various cement-sand ratios used were 1:1, 1:3, 1:4 and 1:6. Dial gauges were used to measure deformation over a gauge length of 200 *mm*.

Based on his experimental studies, Deodhar [28] recommended the use of cement-sand ratio of 1:6 and joint thickness of 10 to 14 *mm* depending upon the fineness modulus of sand to achieve optimum strength of masonry. He has also recommended that frog does not serve any purpose in compression; however, it improves lateral stability. Stress-strain characteristics of brick masonry were also studied.

Ramamurthy *et al.* [71] tested concrete hollow block masonry in running bond for flexural strength normal to bed joints. The influence of full mortar bedding using conventional mortar mixes, the influence of plastering and grouting were investigated by using a bond wrench apparatus. Results showed that the flexural bond strength increases with richness of bedding mortar mix proportion adopted, and observed to be higher than those specified by IS: 1905. The plastering and grouting were found to have significant influence on the flexural strength of masonry, and hence, these aspects need to be included in the design provisions. The characteristic compressive strength of structural masonry is given by the equation:

$$\sigma_{bm} = K\left(\sigma_b \delta_{mf} \delta_s\right)^{\alpha} \left(\sigma_m\right)^{\beta} \qquad \ldots(2.2)$$

where, σ_{bm} = characteristic compressive strength of structural masonry in *MPa*

σ_b = compressive strength of brick in *MPa*

σ_m = compressive strength of mortar in *MPa*

δ_{mf} = moisture factor to account for the moisture content of brick masonry

δ_s = shape factor to account for the shape and size of brick

K, α and β = constants

This equation is based mainly on the results of physical experiments conducted in many countries and is known to give satisfactory results for practical purposes. Pande *et al.* [64] critically examined the above equation from the point of view of theoretical engineering mechanics to investigate as to which mechanical parameters describing both masonry units and mortar influence the compressive strength of structural masonry. They treated masonry as a composite material, with masonry units and mortar joints forming its constituents. Each constituent was assumed to behave as an isotropic linear elastic-brittle material. A two stage homogenization technique was introduced to obtain the average mechanical response of masonry. Based on this, numerical tests with idealized boundary conditions were carried out. It was found that the compressive strength of masonry was not directly influenced by either by the compressive strength of the masonry units or by that of the mortar.

Pande *et al.* [64] observed that the above equation appears to lack rationale from the point of view of engineering mechanics since the parameters that directly influence the compressive strength of masonry are not accounted for in this equation. A new equation was proposed which is too complex for practical use, and hence a set of design charts based on the proposed equation were proposed. The use of design charts was explained through worked examples.

Studies carried out by Gross, Dikkhars and Corogan [39] recommend the use of following equation to obtain the strength of brick masonry:

$$\sigma_{bm} = K_1(2.76 + K_2\sigma_b) \qquad \ldots(2.3)$$

where, K_1 and K_2 are constants in which K_1 = 0.67 without inspection and 1.00 with inspection and K_2 = 0.20 for N-type mortar, 0.25 for S-type mortar and 0.30 for M-type mortar and is as per specification given in ASTM270-1968. The compressive strength of

N-type, S-type and M-type mortar is 5.17 *MPa*, 12.41 *MPa* and 17.24 *MPa* respectively. In India M-type mortar, having compressive strength 17.24 *MPa* and cement-sand ratio of 1:2 is hardly used in masonry. These mortars do not improve the strength of brick masonry considerably.

Experimental studies carried out at Indian Institute of Technology Kanpur (IIT-K), India [31] showed that the compressive strength of brick masonry can be obtained by using following formula:

$$\sigma_{bm} = K\sqrt{\sigma_b \sigma_m} \qquad \ldots(2.4)$$

where, K = a coefficient which depends on layout of bricks and joints.

The value of K is taken 0.275 and 0.303 for loading perpendicular and parallel to joint respectively. This is based on the experimental work carried out on very good quality bricks. Different types of soils are used in different regions for manufacturing of bricks and the values of the constant can not be applied universally. Further, the layout of bricks in masonry construction is fixed depending upon the method of bonding. Masonry is mostly designed to resist vertical loads only, applied perpendicular to the bed joints.

Knutsson [48] carried out experiments which revealed that brickwork built of bricks with identical load carrying capacity but with different performance characteristics, result in different load carrying capacities. Other experiments have revealed that brickwork built of identical bricks, with mortar of the same compressive strength but with different content of admixtures resulted in different load carrying capacities of the brickwork. Traditionally, the load carrying capacity of masonry has been determined by the knowledge of the compressive strength of the Bricks/blocks and the compressive strength of the mortar. Even if the strength of the bricks is denoted by information regarding their water absorption qualities, the strength of the bricks and of the mortar alone is insufficient to predict the strength of the masonry.

Different formulae for predicting the strength of brick masonry proposed by different investigators and discussed above are summarized in Table 2.1.

Table 2.1 Models for Compressive Strength of Brick Masonry

Reference	Model for compressive strength	Values of parameters
Deodhar and Patel [26]	$\sigma_{bm} = \sigma_b - 0.5r'$	r' = sum of ratio of mortar mix
Ramamurthy *et al.* [71] and Pande *et al.* [64]	$\sigma_{bm} = K(\sigma_b \delta_{mf} \delta_s)^{\alpha} (\sigma_m)^{\beta}$	$\alpha = 0.75$, $\beta = 0.25$, $K = 0.40$
Gross *et al.* [39]	$\sigma_{bm} = K_1(2.76 + K_2\sigma_b)$	K_1 = 0.67 without inspection and 1.00 with inspection K_2 = 0.20 for N-type mortar, 0.25 for S-type mortar and 0.30 for M-type mortar
IIT, Kanpur [31]	$\sigma_{bm} = K\sqrt{\sigma_b \sigma_m}$	K= 0.275 and 0.303 for loading perpendicular and parallel to joint respectively
Lenczner D [55]	$\sigma_{bm} = K(\sigma_b)^{\alpha} (\sigma_m)^{\beta}$	$\alpha = 0.50$, $\beta = 0.25$, K depends on fitting

2.4. STATIC BEHAVIOUR OF MASONRY STRUCTURES

The static behaviour of brick masonry structures has been studied by many investigators for observing their response which are presented in subsequent subhead.

2.4.1. Lateral Strength of Brick Masonry Walls

Alshebani and Sinha [1] carried out a series of laboratory tests on half scale brick masonry panels subjected to uniaxial cyclic loading. Forty-two square panels were tested under cyclic loading until failure for two cases of loading: (i) Normal to the bed joint; and (ii) parallel to the bed joint. Failure due to cyclic compression was usually characterized by a simultaneous failure of brick units and head joints or by splitting in the bed joints depending on whether the panel was loaded normal or parallel to the bed joint, respectively. The characteristics of the stress-strain relationships of the two loading conditions were presented in this paper. Envelope, common point and stability point and stress-strain curves were established based on test data, and an exponential formula was found to provide a reasonable fit to the test data. It was concluded that the peak stress of the stability point curve can be regarded as the maximum permissible stress level that was found to be approximately equal to two thirds of the failure stress. It was also observed that the permissible stress level depends on the plastic strain level present in the material due to cyclic loading.

Lee *et al.* [50, 51] introduced a homogenization technique to investigate the elastic-brittle behaviour of masonry panels subjected to incremental lateral loading. For modeling the elastic behaviour of masonry, two successive steps of homogenization were used to obtain equivalent elastic properties. In the first step, brick units were homogenized with vertical joints to give equivalent elastic properties of a stacked system. This stacked system, in the second step, was then homogenized with the bed joints to obtain equivalent material properties for masonry. Tensile cracking was the only non linearity considered in this paper. Cracking was judged on the basis of stresses and strength of each of the constituent materials. The cracks developed, if any, are also homogenized with the homogenized masonry and equivalent non linear stress-strain relationships for cracked masonry were derived. The constitutive model was incorporated in a three dimensional finite element code. It has been verified and validated with experimental data on the response of a set of laterally loaded rectangular masonry panels with and without openings. It was considered that the model can be used for predicting the physical behaviour of laterally loaded panels of arbitrary geometry and boundary conditions.

Thurlimann and Guggisberg [84] tested eight brick masonry panels, loaded with a vertical normal force and transverse bending moments in two directions in a specially designated rig. Thus failure criterion for laterally loaded masonry walls was investigated experimentally.

There has been considerable controversy over which is the most appropriate theory for predicting the transverse lateral strength of masonry walls, which has centered on the methods of analysis, without giving sufficient consideration to the derivation of the material properties. Fried *et al.* [35] compared the results of combining different methods of analysis and material properties to predict wall strength. Actual wall test results from various sources were used in the comparisons. Research into the relationships between various techniques for determining material properties were discussed and the need for researchers to record certain basic properties of the material being used in the masonry research for flexure was emphasized.

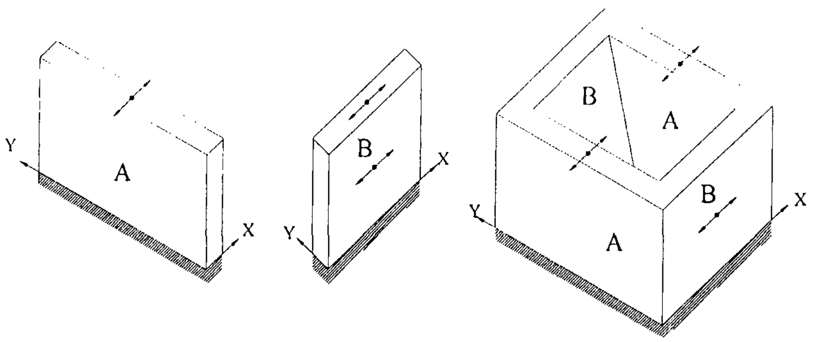
Lawrence and Cao [49] identified and discussed the distinct point of cracking in masonry walls under uniformly distributed out-of-plane lateral loads. Most investigators have concentrated on ultimate load prediction. But this study focuses on the load at which first crack forms. Because this load marks a significant change in behaviour and representing serviceability condition for design, should be studied as a first stage in developing a rational analysis of panel behaviour. A method of analysis based on elastic plate theory for the prediction of loads to cause first cracking in wall panel was presented. A range of practical wall dimensions were covered for supports on three or four sides and the analytical predictions were compared with the results of 32 full scale tests on clay brick walls.

2.5. DYNAMIC BEHAVIOUR OF MASONRY STRUCTURES

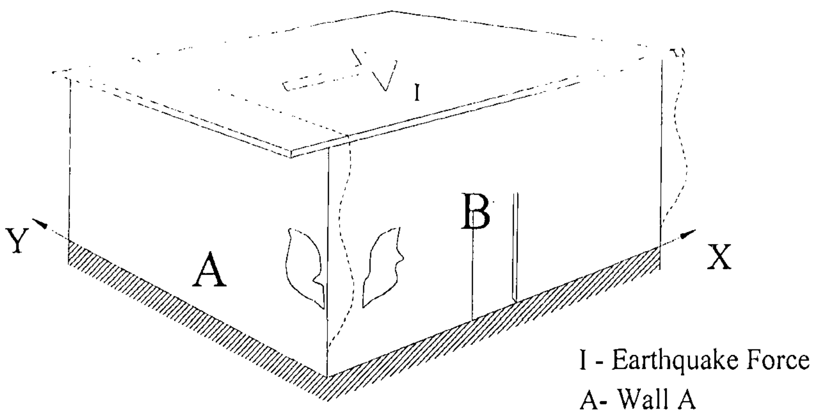
Brick masonry buildings respond to ground motion like all other structures and attract inertia forces depending on their stiffness, mass and damping characteristics [56]. Referring to Fig. 2.1 for the X-direction of motion, walls B act as shear walls while offering resistance against the collapse of wall A as well. Wall A acts as vertical slab supported on two vertical sides and bottom and subjected to inertia force of its own mass. Near the edges the walls have bending moments in the horizontal plane for which brickwork has little strength. Tension on account of vertical bending may generally get relieved due to self weight and can be made to take care of bending tensions. The same will be true for wall B when ground motion is in Y-direction. The roof slab transfers its inertia force at top of the walls causing shearing and overturning forces in them. Major portion of the load is taken up by the shear walls on account of their in-plane stiffness compared to the cross walls. However the slab must have sufficient strength in horizontal plane to be able to transfer the force in the aforesaid manner. Reinforced concrete or reinforced brick slabs would normally possess it, but other type of roof/floor, such as brick tile coverings or timber plank-joist floors, must be connected together and fixed to walls suitably to achieve this goal. Shear walls ofcourse should be able to take the shear of the slab in addition to its own inertia forces and should be designed to be safe for bending and normal stresses resulting from such forces.

2.5.1. Buildings without Base-Isolation

Many investigators including Bruneau [7, 18] investigated the performance of a number of un-reinforced masonry buildings during earthquake in a large portion of North America which were constructed in the absence of mandatory earthquake design requirements, and unquestionably recognized, the type of construction most vulnerable to earthquakes. Awareness of the seismic hazard was relatively new in eastern North America. In addition, the nature of the seismic risk and other engineering constraints shed a new and different perspective on the problem. This state of art paper on the seismic performance of un-reinforced masonry buildings summarized knowledge that has already gained some acceptance in parts of North America, and outlined current limitations, concerns regarding the seismic performance of existing un-reinforced masonry buildings, formulated in an eastern North American seismicity context. The various failure modes of un-reinforced masonry buildings or components subjected to earthquake excitation were described and when possible, illustrated. The state of practice as required by North American building design codes and standards was summarized. A special analytical procedure of the uniform code for building conservation, largely inspired from the Agbabian, Banes and Kariotis (ABK) methodology for the mitigation of seismic hazards in existing buildings was reviewed.



(a) Action of Building Elements



(b) Overall Response

Fig. 2.1 Effect of Ground Motion on a Building

Tomezevic *et al.* [85] summarized the results of an experimental study that investigated the seismic behaviour of two, three-storey, plain and reinforced masonry building models with identical structural configuration. The measured response and observed mechanism of structural behaviour was used to analyze the load bearing and energy dissipation capacity of each structural type. By reinforcing the masonry walls with vertical reinforcement and the border of the walls and horizontal reinforcement in mortar bed joints, the lateral resistance and energy dissipation capacity and global ductility of the building was significantly improved. The mechanism of the behaviour of the tested models changed from the storey mechanism that prevailed in the case of plain masonry model to coupled shear wall mechanism in the case of reinforced masonry building model, with floor slabs and bands contributing to the seismic resistance of the model in the latter case.

Galano and Gusella [36] proposed a design criteria for the seismic reinforcement of the masonry structures by joining shear wall with steel X-bracing. The seismic response of the coupled system was investigated in the time domain by a step by step integration. Two suitable mechanical models for the shear walls and X-bracing were used. A wide parametric analysis was performed varying the mechanical characteristics of the masonry walls as well as the X-bracing. Using 20 simulated ground motions the uncertainties related to the definition of seismic loading were taken into account. The results obtained established a structural design criterion based on the equivalent static lateral force method. Given this criterion, the structural factor for the steel X-bracing had to be reduced. This reduction was necessary due to the mixed matched interaction of the two different seismic behaviours. The reduction was based on a suitable coefficient, named the "coupling parameter". This coefficient was given in specific graphs proposed for design purposes.

Seible *et al.* [75, 76] conducted studies to verify new design guidelines for reinforced masonry buildings in seismic zones, tested a full scale five storey reinforced masonry research building at the University of California, San Diego, under simulated seismic loads. These new design guidelines, developed by the Technical Coordinating Committee for Masonry Research (TCCMAR), limit-state and capacity design principles to ensure ductile seismic response. Design and analysis models developed under TCCMAR for individual components and sub-assemblages were extended to the complete system to predict the seismic response of the five storey research building, and full scale seismic simulation test data were used to provide detailed experimental response data for model verification. The first full scale five storey seismic simulation test under laboratory conditions carried out in U.S.A., showed that even stiff structural wall type masonry building can be designed to exhibit large ductility under extreme seismic loads by controlled inelastic flexural behaviour in predetermined locations, and that analysis and design models exist that can predict the overall seismic response of the complete structural system.

Yan *et al.* [90] developed a comprehensive analytical model and studied the response of un-reinforced masonry under in-plane dynamic loads, including earthquake loads. Masonry was treated as a non-linear homogeneous orthotropic material. A failure envelope was also developed that is capable of predicting sliding and the cracking and/or crushing type of failure. The effect of bed joint orientation was considered, this was achieved through a ubiquitous joint model. The model was capable of performing both static and time history analyses of masonry structures. Non linear dynamic analysis was carried out using the Modified Newton-Raphson iteration scheme in conjunction with the Newmark time integration algorithm. To calibrate the model and to demonstrate its application several numerical examples were treated and the results were compared with those from full scale tests on masonry shear walls under both cyclic and dynamic loads. Reasonably good agreement was found between the analytical and experimental results.

In 1983, Mostaghel *et al.* [58, 59] presented a mathematical model for the response of sliding structures to harmonic motion which was first proposed by Qamaruddin *et al.* [66, 67, 68, 69] for solving problem of single degree of freedom structures supported on sliding substructure and subjected to harmonic and also earthquake support motions. The single storey structures were subjected to 1940 El Centro and 1949 Olympia earthquake ground motions. Spectra for absolute acceleration, relative displacements, sliding displacements and residual sliding displacements were evaluated for three mass ratios, four coefficients of friction and for critical damping of 5%. It was observed from the results of this investigation that for structures with time period less than 1.8 *sec.* the maximum sliding and residual displacements were of order of 1.25 times the peak ground displacements. It was also noted that increase in the levels of input excitations increases the FSI system effectiveness in cutting down the acceleration. However no attempt was made by Mostaghel *et al.* for comparing the analytical results with that of the experimental studies.

2.5.2. Buildings with Base-Isolation

It has now become a great challenge to erect an efficient earthquake resistant structure. So far several methods have been adopted for this purpose but base isolation has become one of the widely accepted techniques [6, 29, 33, 38] which consist of de-coupling the structure from damping effect of horizontal component of ground motion due to earthquakes. There are so many ways, we can separate the super-structure from the sub-structure using base isolation. Seismic base isolation technology is both reliable and cost effective.

Many researchers [42, 43] investigated the various aspects of the seismic performance of base isolated building frames and nuclear power stations, but the results of these studies remain scattered and not easily accessible to practicing engineers. Few other researchers [57, 63] have worked in this field but their contribution is mainly related to conventional reinforced concrete frame buildings. As far as low cost brick masonry structures are concerned there is no study in sight in the literature. Therefore, a literature review has been conducted to assess the state of the art on the performance of conventional masonry buildings and some of them are listed below:

Lin *et al.* [53] proposed base isolation of masonry building, using two sets of mutually orthogonal free rolling rods under the basement of the structure. The result indicated that the super structure response decreases with the decrease in the rolling friction coefficients.

Derham *et al.* [30] in the year 1985, had shown that a building on rubber bearings will be protected simultaneously from unwanted vibration and from earthquake attack. The rubber bearings are suitable for buildings which are rigid and for masonry and reinforced construction upto seven stories. These bearings are very similar to the bearings used in bridges.

The resilient-friction base-isolation (R-FBI) system was proposed by Mostaghel *et al.* [58, 59]. This base isolator consists of concentric layers of Teflon coated plates that are in friction contact with one another and it contains central core of rubber. It combines the beneficial effect of friction damping with that of resiliency of rubber. The rubber core distributes the sliding displacement and velocity along the height of the R-FBI bearing.

Qamaruddin *et al.* [70] studied the behaviour of un-reinforced and partially reinforced masonry house models subjected to earthquake like loads and a new masonry building system with sliding substructure was also presented to obtain the advantage of seismic isolation. Seismic response of masonry buildings with sliding substructure, subjected to Koyna and El-Centro shocks was computed. The results showed that the sliding system scheme was a practical and economical seismic isolation technique to achieve less damage as well as non collapsible masonry buildings requiring only the usual skills of construction locally available.

Fan and Ahmadi [34] analyzed the response to white noise excitation in the frequency domain of a rigid substructure isolated by R-FBI system. The non-linear response was obtained using the equivalent linearization technique, stochastic average method and method of equivalent non-linear system. The resulting mean square responses were compared by performing the Monte-Carlo digital simulation and found to be satisfactory for small values of friction. However, for a higher coefficient of friction, the accuracy of the linearized technique decreases.

Binze *et al.* [18] carried out testing in 1990 for two to six storey reduced scale gypsum models on shaking table to investigate dynamic behaviour of base isolated multistory brick buildings. One of the models had frictional sliding isolation (FSI) scheme whereas the other model was fixed directly on the shaking table. The FSI system was composed of upper and lower reinforced concrete ring beams filled with two layers of asphalt in which graphite powder was interposed. The building models were excited on shaking table with input acceleration varying from 0.1 g to 0.5 g., sliding was initiated at 0.25 g in first model. No damage was seen in the first model up to 0.5 g base acceleration of the table. The second model collapsed at the end of the test.

Tyler [87] was the first to conduct an experimental study on the frictional characteristic of Teflon-steel interface. A similar study was also reported by Constantinou *et al.* [19, 21]. Recently, experiments on Teflon-steel interface have been conducted by Constantinou and co workers [20, 22, 60, 61] to study the frictional characteristics of the Teflon-steel interface under dynamic conditions. The experimental study had shown that the friction between Teflon and steel increases with the increase in acceleration of excitation and decreases with the increase in the bearing pressure.

An experimental study performed to evaluate the feasibility of using a sliding isolation system with uplift restraint devices for a medium rise building subjected to column uplift was carried out by Nagarajaiah *et al.* [62]. The sliding isolation system was found to be quite effective in reducing the structural response and uplift forces, and the uplift restraint of the system was effective in resisting the uplift forces.

Juhn *et al.* [44] presented the results of a series of experiments conducted on shaking table that investigated the response of secondary systems in a sliding base isolated structure. The study indicated that the passive protection offered to secondary systems by base isolation was beneficial, neutral and in some cases detrimental

A shake table test of a six storey quarter scale structure isolated by Teflon disc bearings and helical steel springs was carried out by Constantinou *et al.* [23, 24]. It was found that the system is quite effective in resisting the strong earthquake forces of different frequency content.

Experimental tests by Arya [3, 4, 5] reported the performance of a half size single storey brick masonry building subjected to shock loading in a railway wagon impact facility. Several types of building models were tested, including both isolated and un-isolated, and it was concluded that a building with sliding joints at plinth level performed better than a conventional building.

Kelly *et al.* [45, 46, 47, 77] studied the mechanism of energy absorption in the devices used for resisting earthquake forces and recommended the use of dampers for base isolation to dissipate the energy released by earthquakes.

Izumi *et al.* [40], Jangid [41] and other researchers [52, 54] investigated the behaviour of base isolated buildings and suggested some important measures to be taken into account for the design of base isolation systems.

Poole [65] and Renault *et al.* [72] focused their studies on the use of base isolation against seismic protection in some strategically important buildings.

Su *et al.* [78, 79] carried out comparative study of base isolation systems and investigated the performance of sliding resilient friction base isolation system.

Tadjbaksh *et al.* [80, 81] investigated the response of base isolation as a rigid body and recommended the use of such systems to improve the earthquake resisting capacity of buildings.

Thakkar *et al.* [82] and other investigators [86, 88] on the basis of their research findings suggested a number of protective measures like retrofitting and use of base isolation for the buildings against earthquakes.

2.6. RISK ANALYSIS AND DAMAGE ASSESSMENT DUE TO EARTHQUAKES

Verma *et al.* [89] carried out seismic risk analysis for Kashmir, Punjab, Kmaun and Garwal Himalayan region using all the available seismic data. Assuming that, all events of mag.> 4.5 (*mb*) was detected for the period 1960-199 by the operating seismological networks. A contour map for $1^0 \times 1^0$ areas showing N$\geq$ 4.5 was prepared. This map fairly outlines the areas which was seismically active during this period.

A maximum magnitude map for $1^0 \times 1^0$ areas (period 1950-1990) was also prepared [37]. This figure showed that major events of magnitude greater than 5.0 are located close to the MBF/FCT, two areas transverse to the trend of the Himalaya have also experienced earthquakes of magnitude 5.0 and above. These include Amritsar-Lahore-Faislabad and Delhi-Moradabad region. North of the Indus Suture Zone, major events exceeding magnitude 6.0 were located along the Pamir-Karakoram and the Kun-Lu fault.

Using the maximum magnitude map for $1^0 \times 1^0$ areas, an acceleration map was prepared assuming that the event of this magnitude takes place within a distance of 5 *km*. For this purpose Joyner and Boore's (1981) relationship between maximum acceleration and distance was used. It appeared from this analysis that large part of Himalaya can experience acceleration exceeding 0.20 *g* which was nearly the threshold value for the minimum damage. The analysis showed that major seismic gaps exist in several areas. These were:

- Between Kangra and Garhwal Himalaya, along the lesser Himalaya
- Between Kangra and Kinnaur region in the Higher Himalaya
- Kashmir-Hazara Syntaxial zone, in the Himalayan foot hills

In the NW Himalaya, the present seismic coverage was limited only to Kangra or Chamba region. For proper monitoring of earthquake it was necessary to have seismic stations in Kashmir valley and the Ladakh region.

Arya [2] based on his studies regarding the probable damage due to seismic forces in the U.P. Himalaya concluded that damage potential of moderate earthquake occurring near the centre of each of the eight districts of Uttaranchal taken separately were as given below:

- It will be reasonable to assume the likely occurrence of an M 6.5 earthquake in a district somewhere. The recurrence interval in Uttaranchal area is about 8 years.
- Assumed average distribution of housing and occupancy gives the damage and loss of life estimations vary from 1,10,000 to 352 and 3288 to 510, respectively.
- The damage scenarios worked out herein should serve as base line to start prevention and preparedness activities in each district. The higher authorities of the state as well as the public should be made aware of the prevailing earthquake risk and the role they could and should play to reduce the future earthquake disaster in the Uttaranchal area.

Saikia *et al.* [73] observed that the growth in population together with the infrastructure and lifelines that support urban development along the foothill states has increased the vulnerability of this region to large Himalayan earthquakes. Strategic plans for urban development need to consider seismic safety so that vulnerable structures are retrofitted and new structures are built using reliable design procedures that are based on realistic estimates of the strong ground motions that will occur in future due to large Himalayan earthquakes. Probabilistic seismic hazard maps on both national and regional scales would form a valuable basis for implementation of seismic hazard mitigation measure. Because of the lack of strong motion recordings of large Himalayan earthquakes, it is important that modern seismological methods be used to generate ground motion time histories for such events. The simulated time histories were important for constraining strong motion attenuation relation for crustal earthquakes, especially for the large earthquake magnitudes for which there were practically no recorded data. These time histories were also important for use in the design and analysis of critical facilities such as dams, bridges and important buildings, where it is important to model the non-linear response of the structure in a realistic manner. Experience in the modeling of strong ground motions demonstrates that specific aspects of the earthquake source, wave propagation path and local site conditions frequently dominate the ground motions that are recorded at a site. A significant feature of simulated time histories generated by modern methods was their ability to include in these effects in a specific manner. For example they can include the effects of alluvial river valleys or sedimentary basins, which can strongly influence the amplitude and the duration of strong ground motion. These effects were illustrated in examples taken from the recent Whittier Narrows and Northridge earthquakes in the Los Angeles region in the United States, and the Kobe earthquake in Japan. In each case it was shown how the complex effects of shallow geological structure were manifested in the recorded ground motions, and demonstrated the ability of our ground motion simulation procedures to reproduce these observed effects.

Thus it can be said that a lot of research work have been done regarding the investigation of mechanical properties of masonry, strength of masonry wall panels and seismic performance of plain and reinforced masonry buildings. Seismic hazards have also been investigated and probable damage of building and loss of life has been predicted due to the occurrence of potential earthquake in the earthquake prone areas in India, by a number of researchers. But there is complete disregard as far as investigation of strength and performance of low cost brick masonry structures are concerned which are mainly used by the weaker section of our society.

Chapter 3

EXPERIMENTAL PROGRAM

3.1. INTRODUCTION

The response of load bearing masonry buildings depends upon the basic properties of brick masonry with which it has been constructed, therefore there is a need to establish these properties for the low cost brick masonry used in the construction. The performance of brick masonry buildings under seismic forces also depends upon the lateral load carrying capacity of brick masonry walls, therefore lateral strength of low cost brick masonry used in the construction is also important.

Experiments were planned for determining the basic properties of materials, masonry of different types and to study the response of building models under static and dynamic loading. Besides the testing of the constituent materials, brick masonry prisms were tested to determine the mechanical properties of masonry such as compressive strength, shear strength and stress-strain characteristics. The wall panels were tested under lateral loads to establish lateral resistance of different types of masonry. The building models, one storey high, were tested on shake-table for different vibration characteristics. Sufficient numbers of specimen have been tested for each type of test for ensuring repeatability of test results. The present Chapter contains details of experimental setup, procedure of testing and the experimental results. The statistical parameters for the test results are also given which give an ideal of scatter in the test results.

3.2. MATERIALS USED

3.2.1. Bricks

Half and full size hand moulded burnt clay traditional solid bricks have been used in the present study. The average size of half and full size bricks are 114×57×38 *mm* and 228×114×76 *mm* respectively having frog of sizes 85×30×4 *mm* and 170×60×8 *mm* respectively on one bed face only. The shape of frog was rectangular with rounded corners. These bricks were moulded manually in the laboratory from the brick-earth and were burnt properly in brick-kiln. A schematic view of these bricks is shown in Fig. 3.1. Samples of bricks were tested for crushing strength and moisture absorption as per Bureau of Indian Standards (BIS) specifications [8, 9]. The test results obtained are as follows:

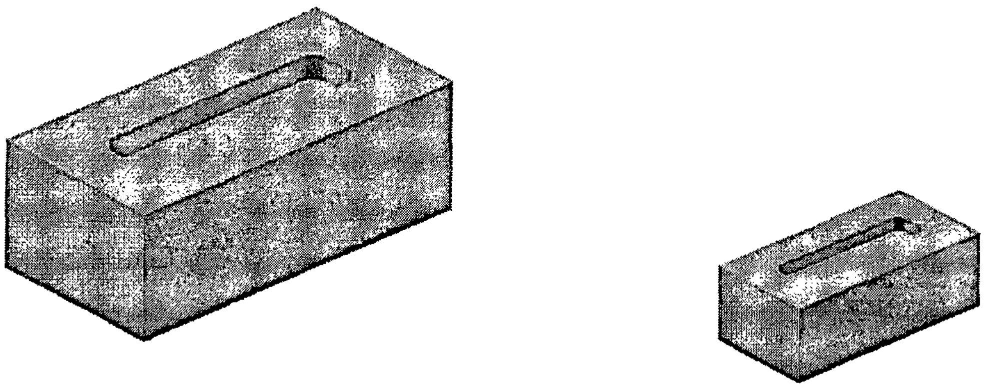
Properties of Half Size Bricks:

Weight of brick	= 0.44 *kg*
Crushing strength on bed	= 8.40±1.55 *MPa*
Coefficient of variation	= 18.40 %
Crushing strength on edge	= 6.60±2.03 *MPa*
Coefficient of variation	= 19.75%
Efflorescence	= Nil
Water absorption after 24 *hrs* immersion in cold water	= 12.60%

Properties of Full Size Bricks:

Weight of brick	= 3.55 *kg*
Crushing strength on bed	= 25.48±4.97 *MPa*
Coefficient of variation	= 19.50%
Crushing strength on edge	= 12.17±2.57 *MPa*

Coefficient of variation	= 21.12%
Efflorescence	= Nil
Water absorption after 24 *hrs* immersion in cold water	= 11.53%



(a) Full Size Brick (b) Half Size Brick

Fig. 3.1 Schematic View of Bricks Used

3.2.2. Cement

Ordinary Portland Cement (OPC) was used in the preparation of masonry prisms. The cement was tested for the normal consistency, compressive strength, initial and final setting time as per BIS specifications [10, 11]. The total quantity required was procured in one instalment so as to ensure uniformity. The results obtained are as follows:

Compressive strength (28 *days*)	= 29.70 *MPa*
Normal consistency	= 30%
Initial setting time	= 40 *min*
Final setting time	= 8 *hrs*

3.2.3. Sand

Locally available river sand was used. The sieve analysis was carried out and percentage passing through different sizes of sieve has been plotted in Fig. 3.2. The grading of sand projection on either side and the frog of both of the bricks was upward. The specimen was tested under Universal Testing Machine as shown in Fig. 3.3.

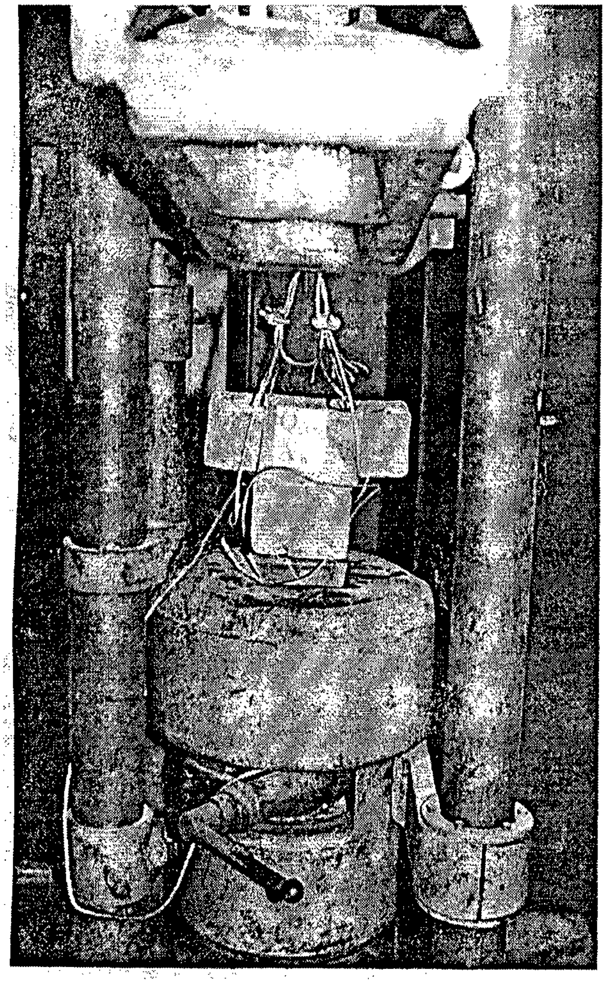


Fig. 3.3 Testing of Specimen for Bond Strength

The same mason prepared all the prisms, wall panels and building models so as to achieve uniform workmanship. The thickness of mortar was kept uniform at $D/6$, where D is the thickness of brick, thus its value for model bricks was 6 *mm* and that for the full size bricks was 12 *mm*. The properties of the mortar used were as follows:

Compressive strength at 28 *days*	= 7.11±0.75 *MPa*
Coefficient of variation	= 10.66%
Tensile strength at 28 *days*	= 0.78±0.06 *MPa*

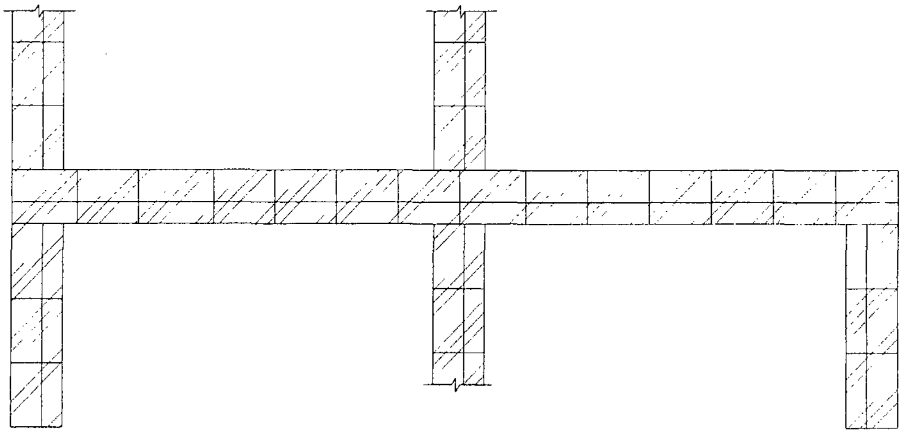
Coefficient of variation	= 7.76%
Bond strength	= 0.14 MPa
w/c ratio	= 0.80
Density	= 1920 Kg/m^3

3.3. TYPES OF BRICK MASONRY

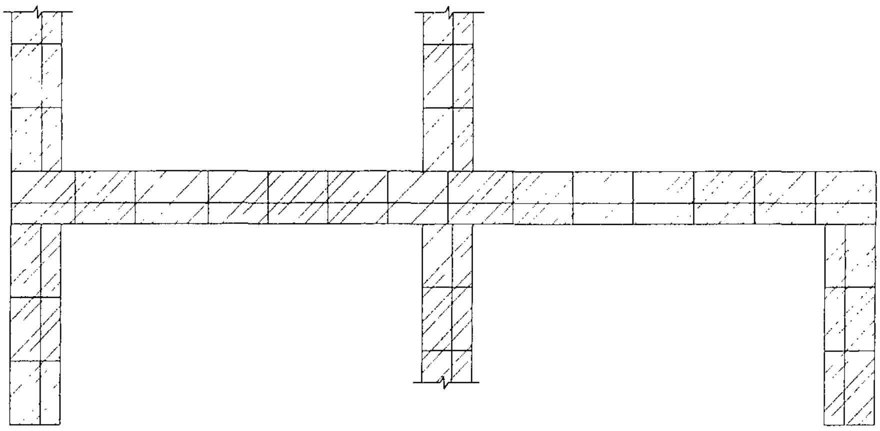
For the purpose of identification, the prisms were classified into four groups. Each group consists of twelve samples, out of which, six samples each were tested for compressive and shear strength. The classification is given in Table 3.1. Three low cost options viz., (i) Type B: reduced thickness, (ii) Type C: odd course of bricks on edge and even course on bed; and (iii) Type D: bricks on edge in Flemish bond were studied. Besides these three low cost brick masonry types, conventional brick masonry i.e. Type A masonry in Flemish bond was also taken for the study.

The arrangement of bricks in different courses of brick masonry in the construction of walls and its junctions for the three types of low cost brick masonry are shown schematically in Figs. 3.4 to 3.6. In Fig. 3.4, two course numbers are specified (viz. I-I, II-I/II, III-II), first one is for bricks on bed, whereas the second course number is for bricks on edge. It is to be noted that in the second course of bricks on bed, there are two courses for bricks on edge (half from each course) due to which it is written as: II-I/II Course.

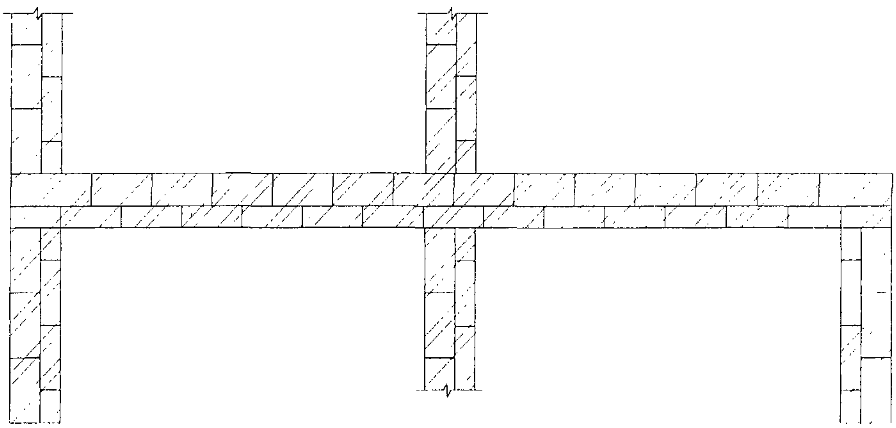
The construction of brick masonry walls showing different junctions in the conventional (Type A) and the three low cost brick masonry types (Type B, C and D) are shown in Fig. 3.7.



(a) I-I Course

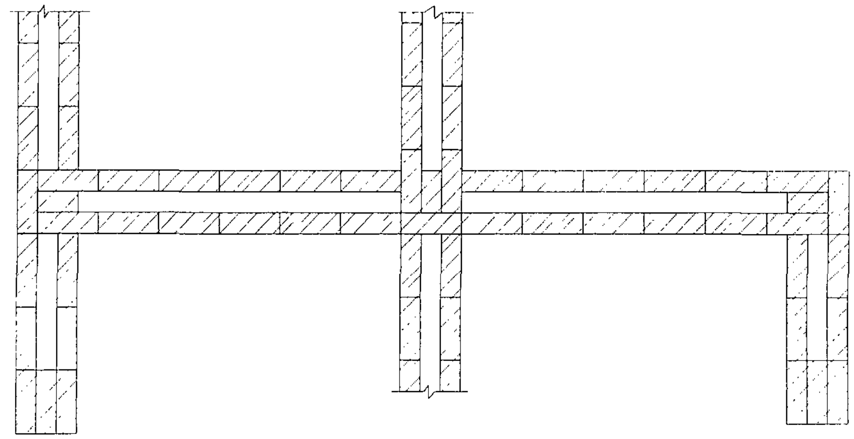


(b) II-I/II Course

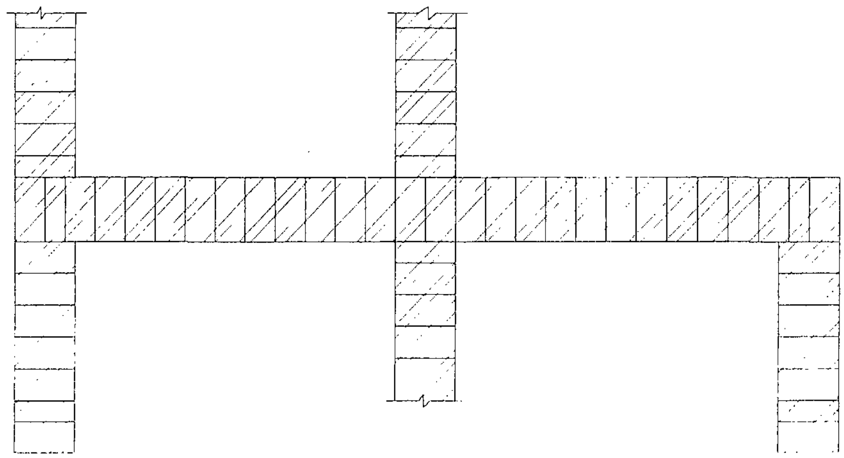


(c) III-II Course

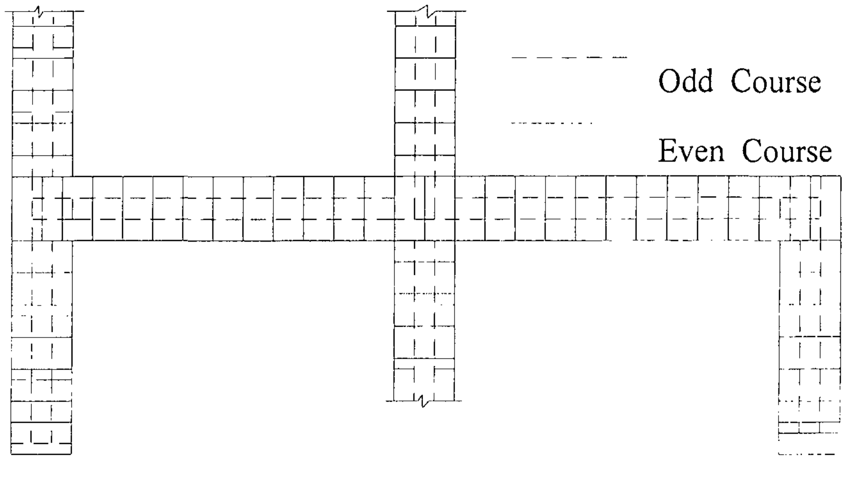
Fig. 3.4 Construction of Wall using Masonry Type B



(a) Odd Course

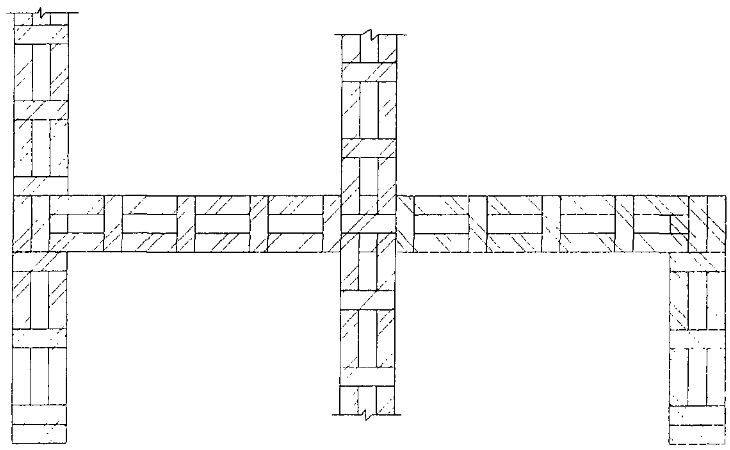


(b) Even Course

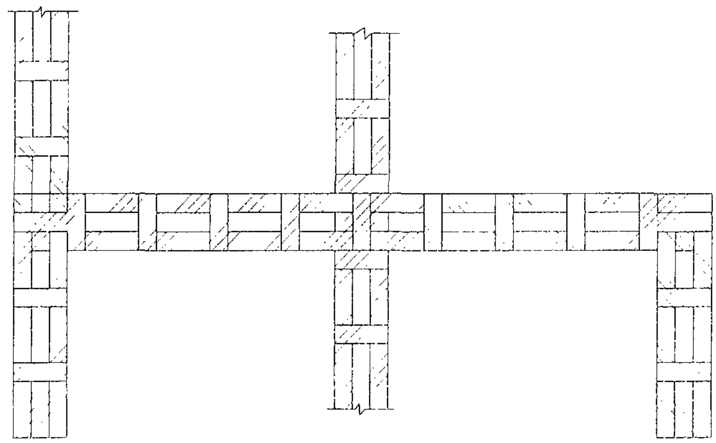


(c) Odd/Even Course

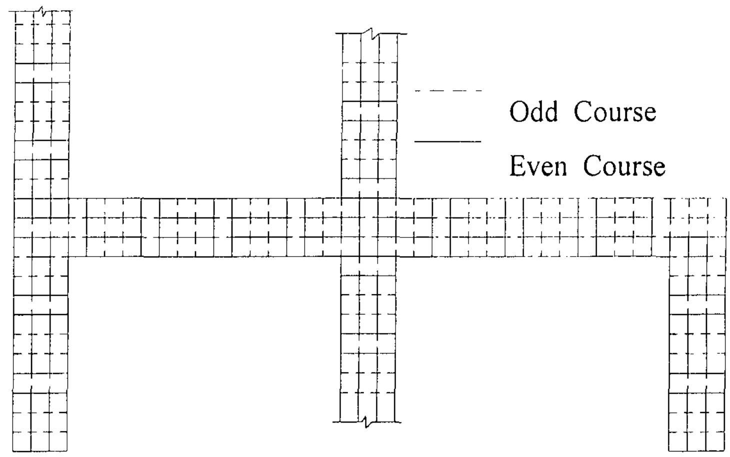
Fig. 3.5 Construction of Wall using Masonry Type C



(a) Odd Course

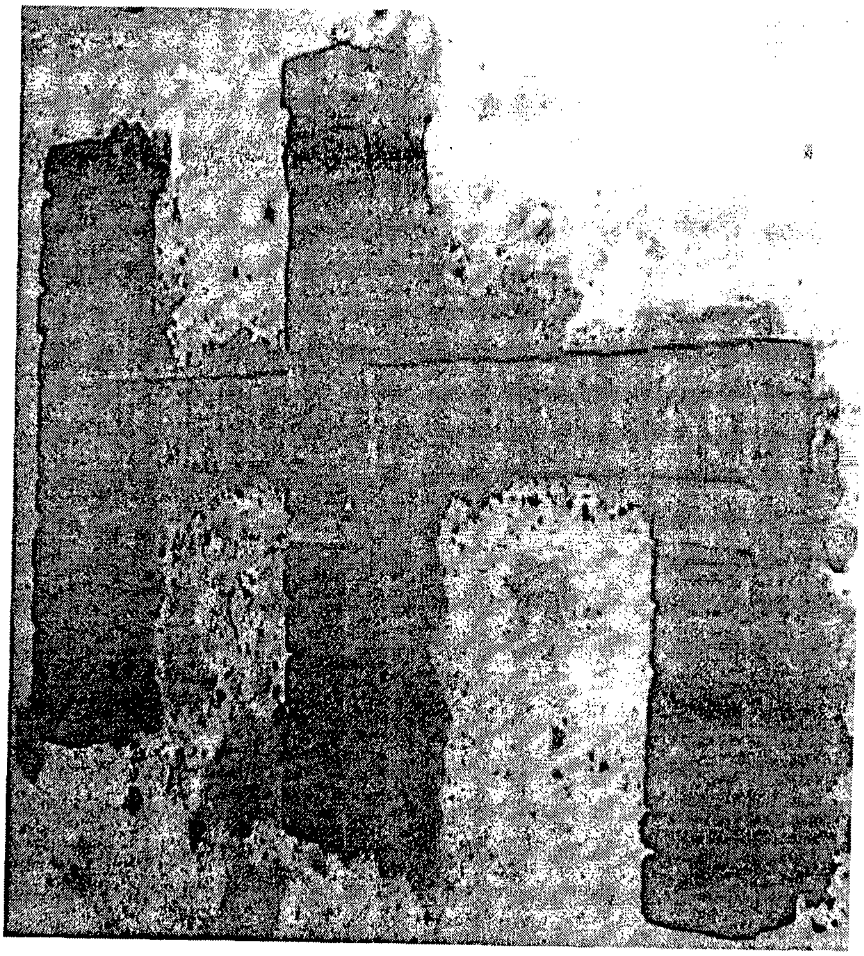


(b) Even Course

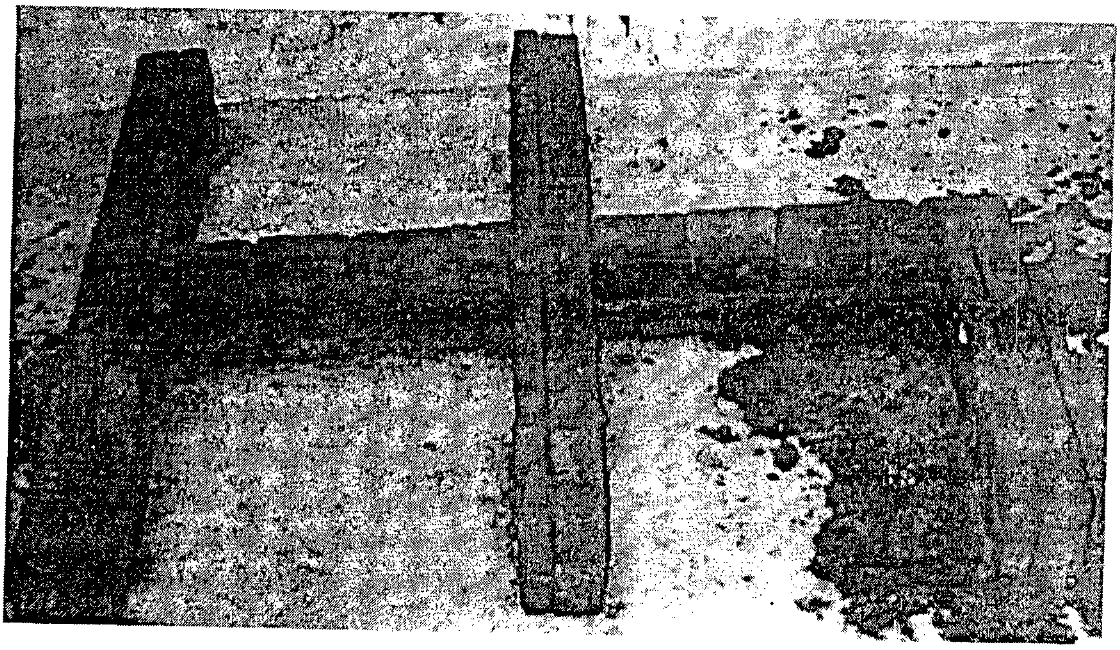


(c) Odd/Even Course

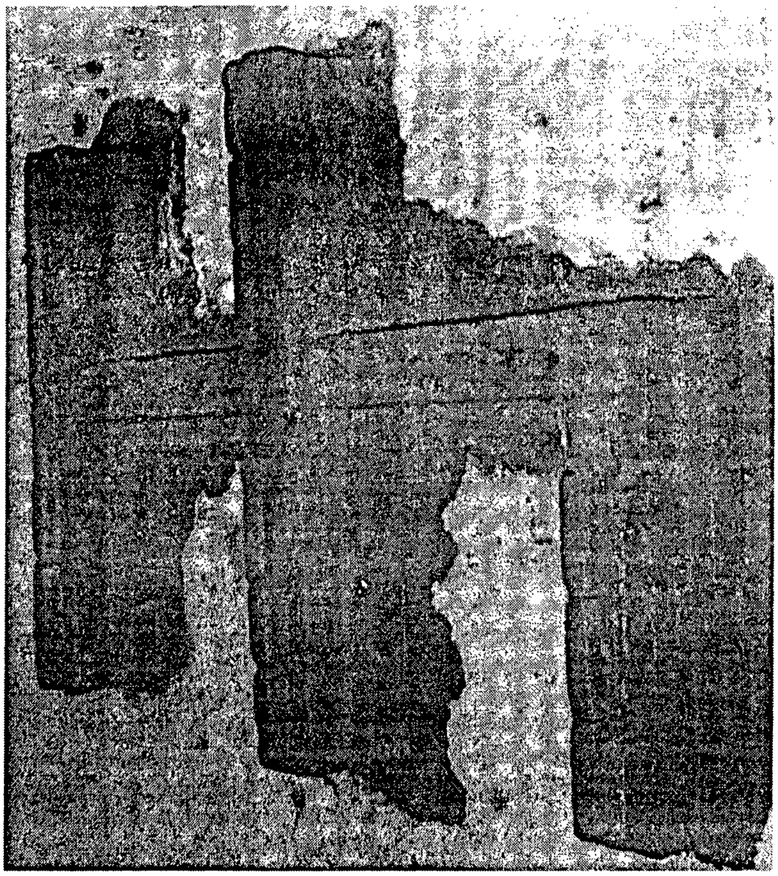
Fig. 3.6 Construction of Wall using Masonry Type D



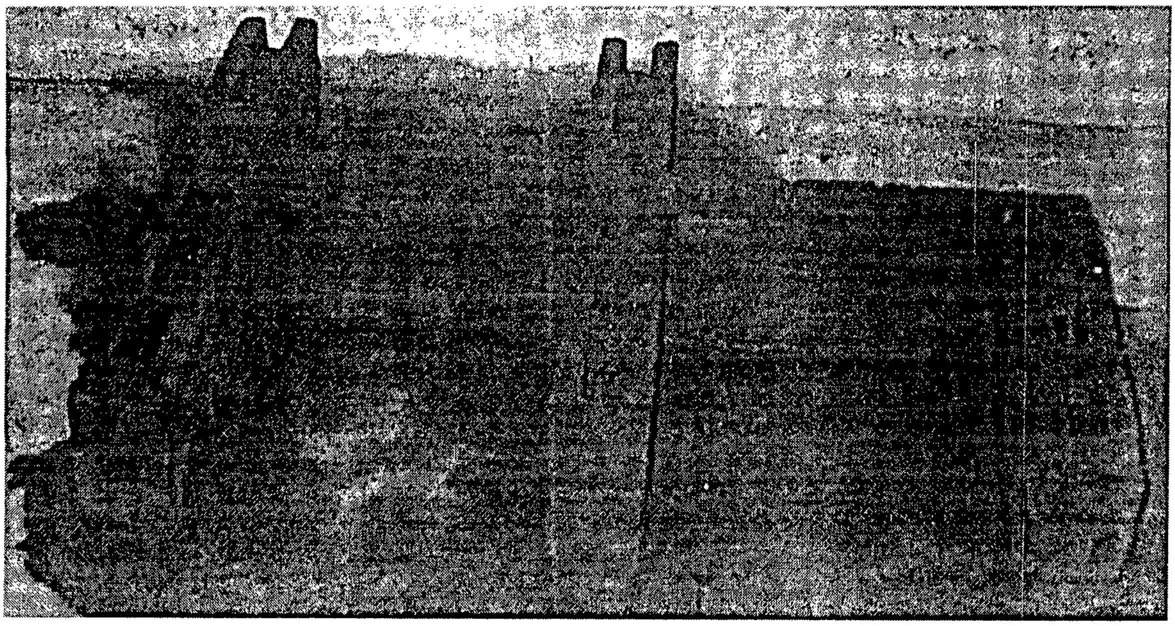
(a) Masonry Type A



(b) Masonry Type B



(c) Masonry Type C



(d) Masonry Type D

Fig. 3.7 Arrangement of Bricks in Different Courses and Junctions of Masonry Walls

Table 3.1 Types and Specifications of Brick Masonry Used*

Group	Type of masonry	Arrangement of bricks	Size of prisms $B_1 \times B_2 \times h$ (mm)	h/B_1	Number of courses
A	Solid	Bricks on bed	114×114×260	2.28	6
B	Solid	Bricks on bed and on edge in every course	100×114×260	2.60	6 on bed and 4 on edge
C	Hollow	Odd course on edge and even on bed	114×114×317	2.78	6 3 on edge and 3 on bed
D	Hollow	Bricks on edge	114×203×375	3.28	6

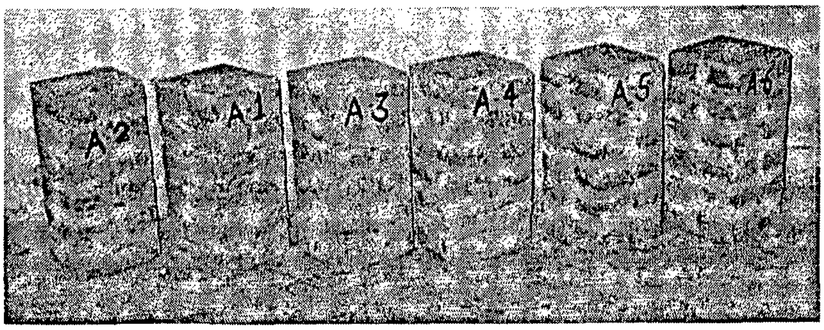
*B_1 and B_2 are the lateral dimensions of prism ($B_1 \leq B_2$) and h is the height of the prism.

3.4. BRICK MASONRY PRISMS

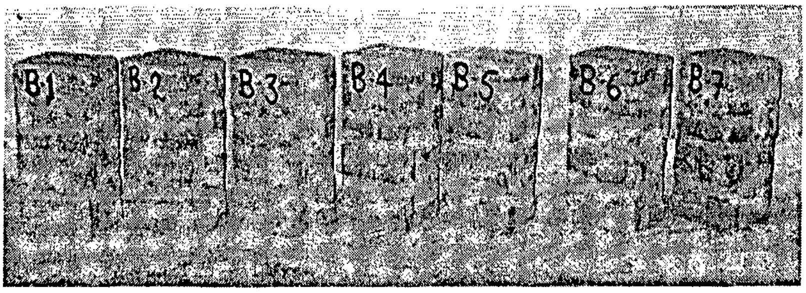
Twelve prisms were made for each type of masonry construction and six each were tested in compression and shear. The sizes of brick masonry prisms of different types of masonry are different because of different arrangement of bricks in each type. The length and height of prism is taken as the minimum that may represent a particular type of masonry.

3.4.1. Preparation of Prisms

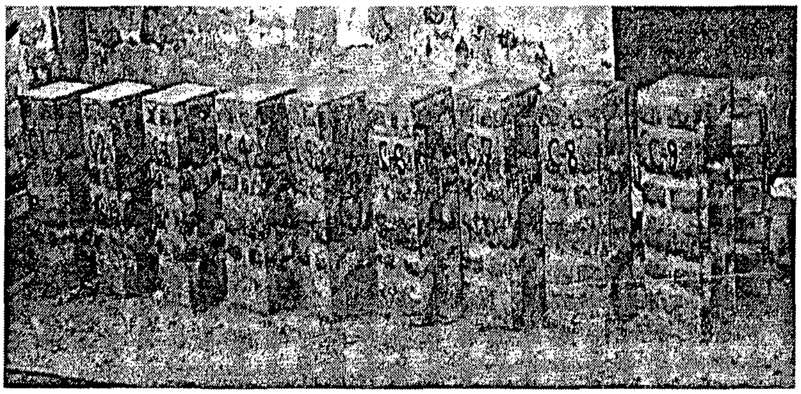
Bricks were soaked in water completely for one *hour* before laying. The joints in the masonry were filled with a uniform thickness of mortar as far as possible. Mortar was used within 30 *min* of adding water. During the making of prisms, the sides of prism were checked for verticality. The prisms were kept under wet gunny bags after 24 *hrs* after their making and were cured by sprinkling water for 10 *days* [15]. The various types of masonry prisms prepared for testing are shown in Fig. 3.8.



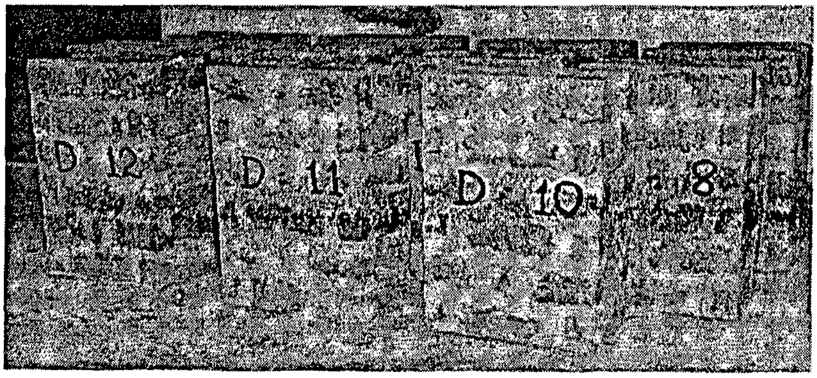
(a) Masonry Prism Type A



(b) Masonry Prism Type B



(c) Masonry Prism Type C



(d) Masonry Prism Type D

Fig. 3.8 Different Types of Masonry Prisms

3.4.2. Testing of Prisms in Compression

Before testing, the prisms were air dried for 24 *hrs* and Demac points were fixed on the prisms with the help of Araldite at a distance equal to the gauge length of Demac gauge, placed centrally along the height of the prisms. The length, width and height of each prism were measured to the nearest *mm* before testing. The dimensions of prisms with their standard deviation are given in Table 3.2. The low value of standard deviation shows uniformity in their making. The prisms were placed between 10 *mm* thick MS plates weighing 2.90 *kg* each, one at top and the other at bottom surface of prisms. This was done to achieve uniformity of loading on test specimens. Two dial gauges were placed at suitable position on universal testing machine for measuring change in the total height of prisms. The prisms were then tested to failure in universal testing machine.

The load was applied gradually and the dial gauge and Demac gauge readings were taken at suitable load intervals. The shortening was measured simultaneously by dial gauge and Demac gauge which was held in position by hand. The test setup for the testing of brick masonry prisms in compression is shown in Fig. 3.9.

Table 3.2 Dimensions of Prisms with Standard Deviation

Masonry Type	B_1 (*mm*)	B_2 (*mm*)	*h* (*mm*)
A	114±1.5	114±1.5	260±1.7
B	100±1.1	114±1.4	260±1.8
C	114±1.50	114±1.4	317±1.9
D	114±1.4	203±1.6	375±2.1

Since the h/B_l ratios of all the four type of masonry prisms were different, hence the ultimate compressive strength of the prisms tested was multiplied by a correction factor. The correction factor for h/B_l ratio recommended by BIS [16] for solid brick masonry prisms (i.e. Type A), given in Table 3.3, has been taken as a guide. The corresponding values of slenderness ratio of prism are also reported in the Table.

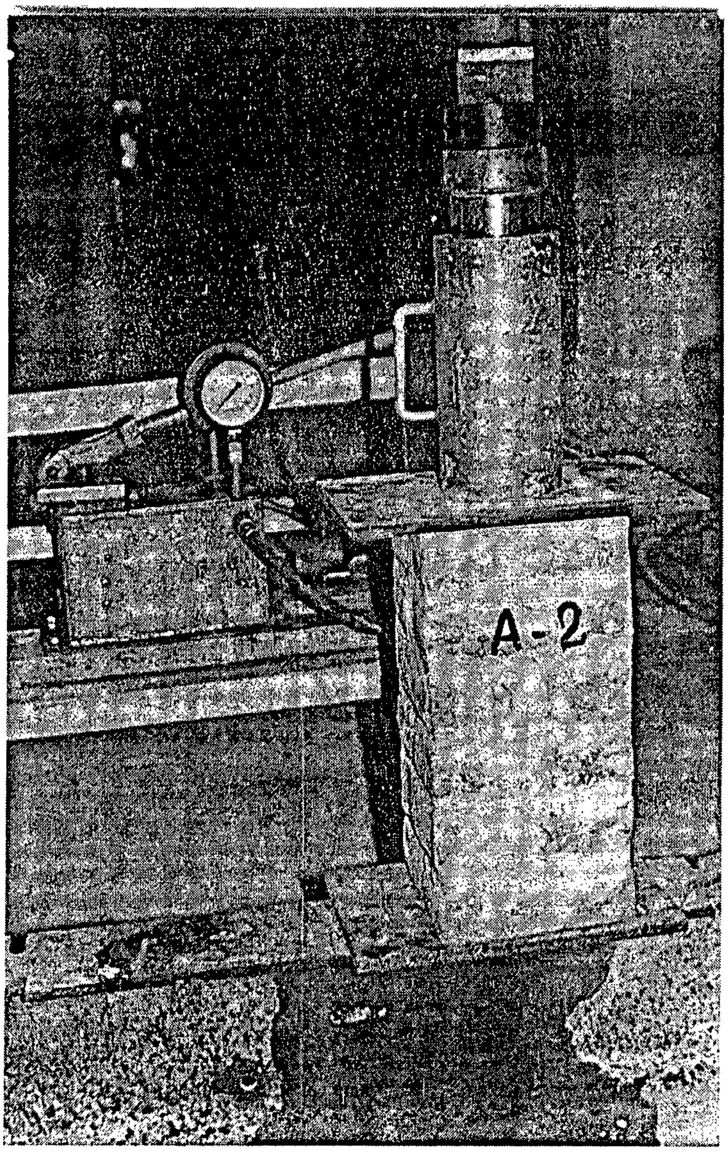


Fig. 3.9 Test Setup for Brick Masonry Prism in Compression

The same correction factors have been used in different types of masonry taking the slenderness ratio as the basis. The values of minimum radius of gyration of different types of masonry prisms found from the analysis are:

$$r = K_r \cdot \frac{B_1}{2\sqrt{3}} \qquad \ldots(3.1)$$

where, r = minimum radius of gyration

B_l	= least lateral dimension of masonry prism
L	= length of brick
B	= width of brick
D	= thickness of brick
K_r	= coefficient for radius of gyration

$= 1.0$ For masonry Type A and B

$$= \frac{0.56B + 0.85D}{B + D} \quad \text{For masonry Type C} \qquad \ldots(3.2)$$

$$= \sqrt{\left(\frac{0.20}{L} + \frac{0.05}{D}\right) B_1} \quad \text{For masonry Type D} \qquad \ldots(3.3)$$

Table 3.3 Correction Factor for Type A Brick Masonry Prism

$\frac{h}{B_1}$	2.0	2.5	3.0	3.5	4.0	5.0
Slenderness ratio, $\frac{h}{r} = 2\sqrt{3}\,\frac{h}{B_1}$	6.9	8.6	10.4	12.1	13.8	17.3
Correction factor, C_f	0.73	0.80	0.86	0.91	0.95	1.00

The correction factors may also be obtained from the following equation:

$$C_f = -0.02\left(\frac{h}{B_1}\right)^2 + 0.23\left(\frac{h}{B_1}\right) + 0.35 \qquad \ldots(3.4)$$

or, $$C_f = -0.002\left(\frac{h}{r}\right)^2 + 0.066\left(\frac{h}{r}\right) + 0.353 \qquad \ldots(3.5)$$

The correction factors found for different types of masonry based on their slenderness ratio are given in Table 3.4.

Table 3.4 Correction Factors for Different Types of Brick Masonry

Masonry type	Height of prism, h (mm)	Minimum radius of gyration, r (mm)	Slenderness ratio, $\frac{h}{r}$	Correction factor, C_f
A	260	32.9	7.9	0.77
B	260	28.8	9.0	0.81
C	317	22.2	14.2	0.96
D	375	19.4	19.3	1.00

3.4.3. Testing of Prisms in Shear

An arrangement was made for testing of masonry prisms in shear [74]. This arrangement consisted of a sealing wire, clamp, nut-bolt and a flat iron strip bent to rectangular/square shape having its length and width equal to that of the prism to be tested. The sealing wire was fixed in the jaws of movable part of universal testing machine. This sealing wire was made to pass through a pulley, which was placed to the fixed part of universal testing machine with the help of a flat iron bar.

A flat iron strip was bent to a rectangular/square shape, with its sides equal to that of the prism to be tested. One side of the bent iron strip was made to project slightly, with projected part having hole. To this hole, was inserted another end of sealing wire. Steel vice was fixed on a compression-testing machine, located at a distance of 4 m from universal testing machine. The prism was placed in the steel vice which was tightened properly. The rectangular/square shaped bent iron strip was fixed at one of the courses of the masonry prism, just above the mortar joint and tightened. The sealing wire passing through the rectangular/square shaped bent iron strip was tightened properly with the help of clamped nut-bolt. A suitable load, obtained by scaling, was placed on the top surface of the prism.

The universal testing machine was operated and the moving part of it started moving upward. Load was increased gradually and this load was transferred laterally to the prism with the help of sealing wire and pulley arrangement. The load was increased till the prism failed in shear. The load at which prism failed in shear was noted down. The test setup for brick masonry prism in shear is shown in Fig. 3.10.

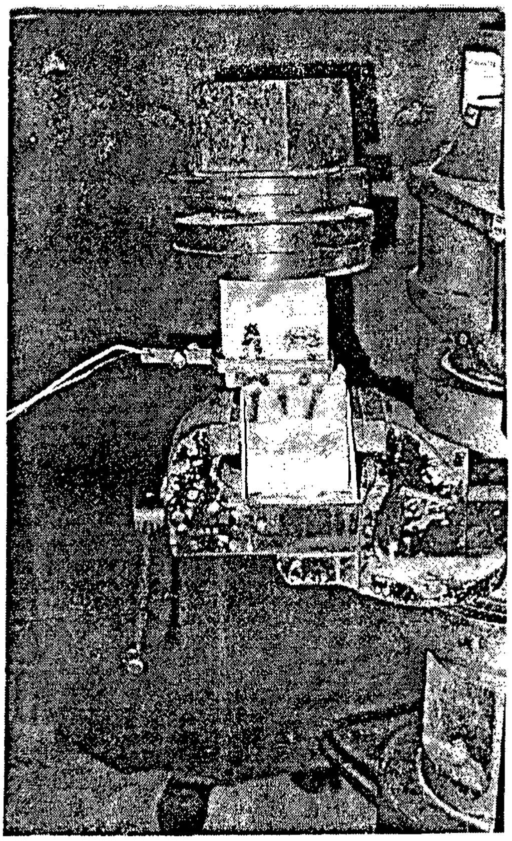


Fig. 3.10 Test Setup for Brick Masonry Prism in Shear

The masonry Type C was not taken for further study due to its very low compressive and shear strength.

3.5. MASONRY WALL PANELS

The wall panels of size 1200×1000 *mm* were constructed using masonry Type A, B and D with full size bricks. The construction of wall panels was in accordance with the specifications of IS: 2212-1962 [15]. The wall panels were checked for verticality during construction. The thickness of the wall panels built with three types of masonry A, B and D were 228, 190 and 228 *mm* respectively. The three exposed edges (two sides and top) of masonry Type D were having cavity as in the rest of the portion of the wall i.e. no closure course was used at the free edges. These walls were built on steel channel ISLC 300 @ 33.1 *kg/m*. All the exposed surfaces of the walls were kept un-plastered. Only the bottom edge of the wall panels was fixed and the other three edges were free.

3.5.1. Testing of Wall Panels

The wall panels were cured for 10 *days* before testing [15]. A horizontal force was applied at the top of the wall with the help of a hydraulic jack of 100 *kN* capacity having a least count of 0.5 *kN*. A rectangular plate of 200 *mm* height and width equal to the width of wall panel was used to transmit the lateral load from hydraulic jack to the wall panel. The load was applied in increments of 0.5 *kN* till the failure of the wall panel. The load was in plane and horizontal throughout the test [17].

Three dial gauges was placed on the vertical edge opposite to the loaded edge of the wall – one each at the top, mid height and bottom. The dial gauge affixed at the bottom was used for checking the fixidity of the wall panel at the base. Four sets of Demac gauges named D1 to D4 were affixed, two each along both the diagonals. The readings of dial and Demac gauges were taken at each load increment and crack patterns were also recorded. The test setup for wall panel under static lateral load is shown in Fig. 3.11.

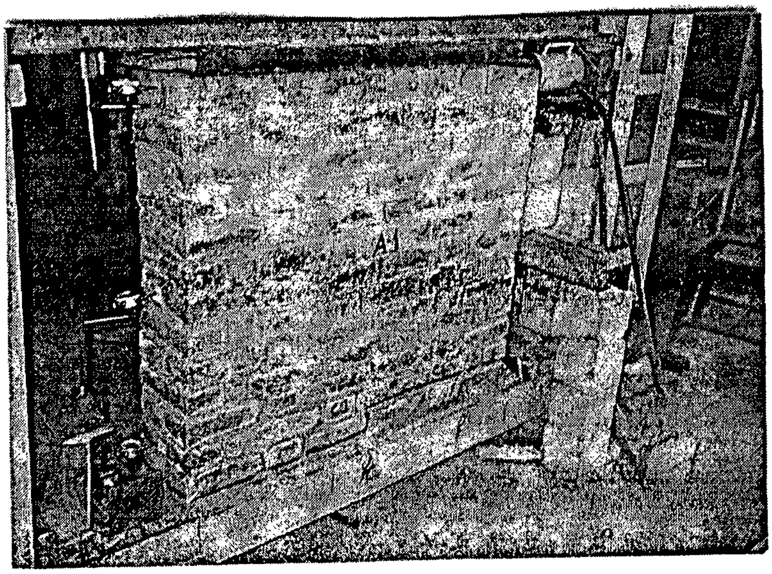


Fig. 3.11 Test setup for Wall Panel under Static Lateral Load

3.6. MASONRY BUILDING MODELS

The present study was confined to a single room building model which has simple plan and symmetry. The reasons for its selection are: (i) it has greatest chances of survival; (ii) it is easy to understand the behaviour of the structure; and (iii) it minimizes torsion and stress concentration. Moreover, longer buildings are likely to experience greater variation in ground movement and soil conditions. By keeping the length of building model less than three times its width, movement generated due to differential movement of ground can be minimized. The building model thus selected for testing was single room with aspect ratio close to unity.

The building models having single room were constructed with half size model bricks (114×57×38 *mm*) using 1:6 cement-sand mortar with w/c ratio of 0.80. During construction, the sides of the walls were checked for verticality. The size of building models was 1070×990×660 (high) *mm*, thickness of cast-in-situ reinforced concrete slab was kept as 40 mm reinforced with 5ϕ @ 200 *mm* c/c bothways. A double leaf wooden door was fitted in the middle of one of the longer walls and a double leaf glass window pane with wooden panel was used in the other longer wall. The direction of load/vibration was parallel to the longer walls. The constructed building models were kept under wet gunny bags for 24 *hours* and thereafter cured for 10 *days* [15].

These building models were constructed on 1100×1000 *mm* rectangular frames of rolled steel channel section ISJC 100 @ 5.8 *kg/m*. The specifications of the channel section and size of frames used for doors and windows are as given in Table 3.5.

Table 3.5 Specifications of Doors and Windows

Item	Door	Window
Type	Wooden double leaf	Wooden double leaf with glass pane
Size	340×210 *mm*	200×120 *mm*
Thickness of panel	20 *mm*	10 *mm* (glass pane)

Door and windows were fitted in the equal angle (25.4×1.5 *mm*) section frame. Windows were anchored in the walls by 50 *mm* extension of angle at all the four corners and doors were anchored only at the top corners.

3.6.1. Experimental Setup and Testing under Static Loads

The models prepared for studying the performance of low cost brick masonry buildings under static load, were named as HM-1, HM-2 and HM-3 for the three types of masonry i.e. Type A, B and D respectively. For the testing of building models an arrangement was made in the laboratory. The building model was placed between the two strong steel columns of testing frame and the base of the model was fixed so that it may not move as a whole during testing. The hydraulic jack was placed at the roof level between the column of the steel frame and the model. The direction of load was parallel to the walls in which door and window was installed. An angle of length equal to that of model was placed between the hydraulic jack and roof level so as to transfer the load uniformly to the whole width of the model [25].

The points for recording the deflection of building were marked on the opposite cross wall. There are two points at the top level, two at the bottom and one at the centre of the wall. The dial gauges were fixed at the desired position with the help of a suitable arrangement of steel girders. The diagonals on the shear walls were marked. On these diagonals, Demac points were fixed with the help of Araldite for the two positions of Demac gauge at top side and two positions at bottom side to record the diagonal compression and tension in the two diagonals. Similar arrangement was there on the opposite wall.

The load was applied with the help of hydraulic jack at an interval of 0.5 *kN*. The measurements of deflections were taken by the dial gauges and diagonal compression and tension was measured with the help of the Demac gauges. The cracks developed on different walls during loading were marked. The testing was continued till the failure of building i.e. no further cracks were developed inspite of the increase in the magnitude of the load. The setup for building model under static load is shown in Fig. 3.12.

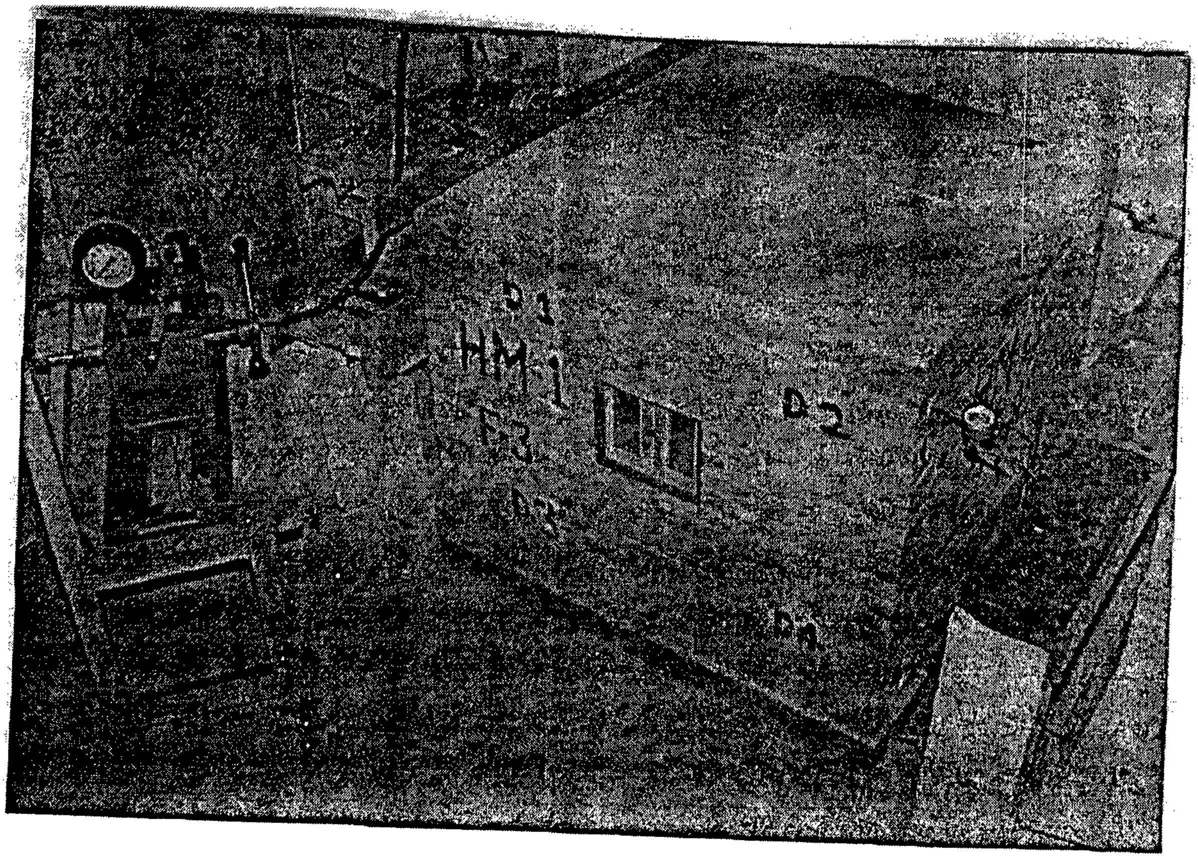


Fig. 3.12 Test Setup for Building Model under Static Lateral Load

3.6.2. Experimental Setup and Shake Table Testing

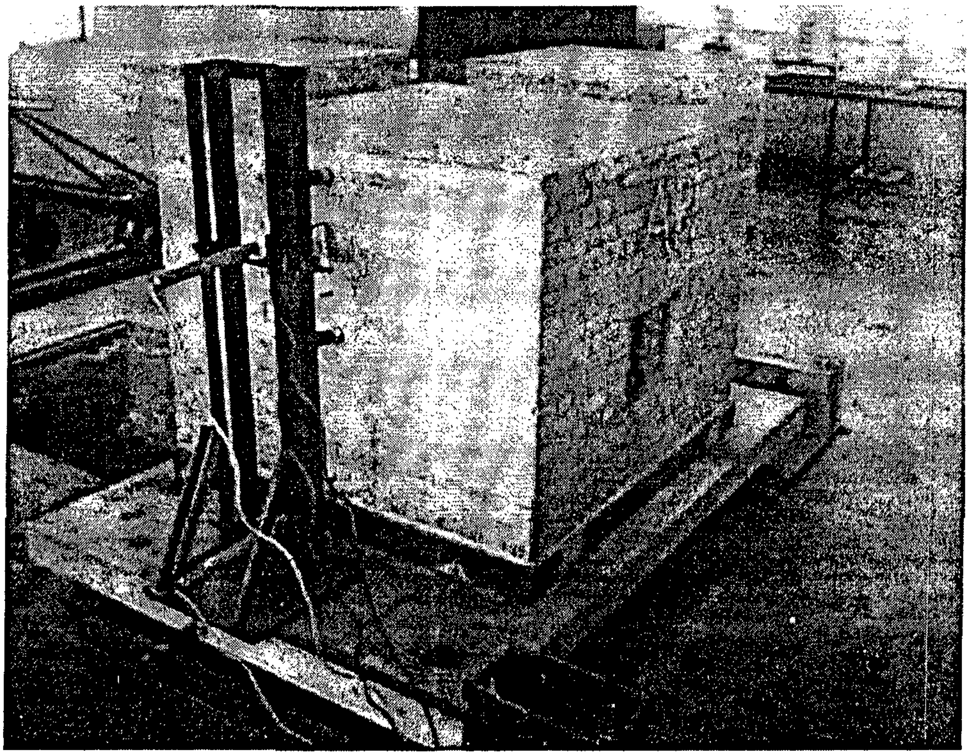
Dynamic testing of building model was carried out in the Laboratory. The specifications of the shake table are as follows:

Size of platform:	1828×1219 *mm*
Type:	Rolling ball type
Capacity:	900 *kg*
Dampers:	Open coil helical springs

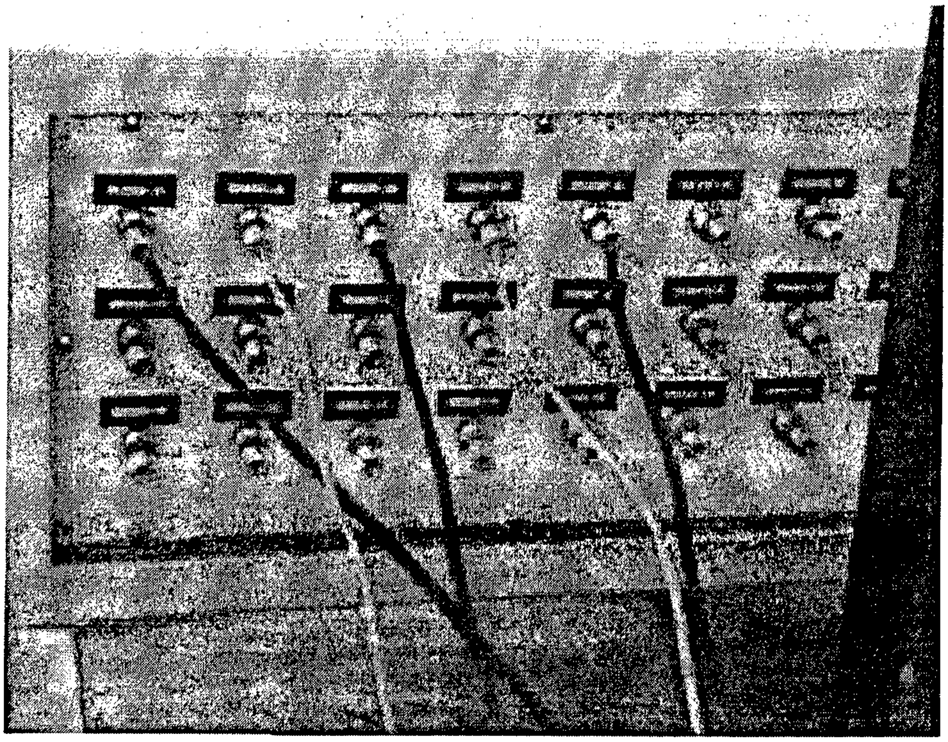
T-6403

Using this shake table, a model may be tested under uniaxial horizontal vibrations. To generate vibrations, mechanical oscillator (Lazan type) with eccentric mass (100 *kg*) is used. Eccentricity of the mass may be increased from 0 to 140 *deg* in a step of 4 *deg*. Mechanical oscillator is driven by a 5 *hp* D.C. motor. The speed of the motor may be upto 3000 *rpm* in steps of 3 *rpm* which is controlled by a speed control unit. The building models were tested for speeds 300–3000 *rpm* in steps of 300 *rpm*. The magnitude of force increases with the increase of eccentricity of mass and speed of motor. A peak force of 1800 *kg* is reached at a speed of 3000 *rpm* and an eccentricity of 140 *deg*. To record the data obtained from the dynamic testing of building models, a data logger with 48 channels which is compatible with a computer is used. With the help of this facility as many as 1000 samples per *min* may be recorded. Linear variable displacement transducers (LVDTs) were used to record displacements and piezoelectric accelerometers were used to record acceleration. LVDTs are sophisticated enough to record displacements upto ±40 *mm* with a precision of 0.01 *mm* and capacity of accelerometer is $\pm 5g$.

Three building models, each constructed with different types of brick masonry (Type A, B, and D), were tested with and without base isolation. Teflon sheet, chemically known as poly tetra flouro ethylene (PTFE) was used for the purpose of base isolation. Base isolation was provided at the plinth level covering the entire length and thickness of the walls. The direction of motion of shake table is parallel to the longer walls. Two accelerometers, one each at top and middle of a short wall of building models and one at the shake table were used to record the acceleration. Two LVDTs, one each at the top and base of the short wall of building models were used to record the displacement of building models. Both the LVDTs and accelerometers lie in the same vertical line passing through the mid length of the short wall. The entire arrangement of the test setup used for the shake table testing of building models is shown in Fig. 3.13.



(a) Building Model Mounted on Shaking Table



(b) Data Acquisition System

Fig. 3.13 Test Setup for Shake Table Testing of Building Model

Chapter 4

EXPERIMENTAL OBSERVATIONS AND ANALYSIS

4.1. INTRODUCTION

The analysis of data and observations made in experiments conducted on brick masonry prisms, wall panels and building models are presented in this chapter. The test procedure of different tests conducted is explained earlier in Chapter 3.

The mathematical models have been developed for the estimation of basic properties for different types of brick masonry constructed with half size bricks. The relationship has been established for predicting the properties of masonry with full size bricks which may be used for design purposes.

The mathematical models have been developed for the estimation of lateral stiffness of wall panels. Finite element analysis has been carried out for the analysis of wall panels using standard software package for validating their performance under static lateral load. The basic properties obtained for different types of brick masonry have been used in the analysis. The building models tested for dynamic load with and without base isolation have been analyzed considering the building as a single degree of freedom system.

4.2. MECHANICAL PROPERTIES

The mechanical properties of brick masonry such as compressive strength, shear strength, stress-strain characteristics and modes of failure of different types of brick masonry normally employed in low cost housing have been determined through detailed experimental investigation. Mathematical models for these properties have also been developed. As the size of brick used in the construction of models was half of the size of brick normally used in the prototype structures, therefore, a relationship has also been proposed for extrapolating the results of models to the prototype. It is with this objective that the masonry prisms with full size bricks were also tested for establishing relationship between the masonry prepared with half and full size bricks.

4.2.1. Stress-Strain Characteristics

The load-deflection curves and stress-strain (actual and nominal stress) curves for each type of masonry prism tested under vertical axial compression have been plotted in Figs. 4.1 to 4.3. The actual stress has been calculated on the basis of net area of cross-section of prisms, whereas the nominal stress is the stress calculated on the basis of the gross cross-sectional area (i.e. outer area of cross-section) of prism. The actual and the nominal stresses are same for masonry Type A and B which are solid, whereas for masonry Type C and D, these stresses will be different due to cavity inside the prism. The load carrying capacity of masonry wall per unit length can be calculated by multiplying the ultimate nominal stress with the thickness of wall, whereas it may also be calculated by multiplying the ultimate actual stress with the net thickness of wall. The compressive strain used for plotting the stress-strain curves is the strain measured with the help of Dial gauges. The Demac gauge readings have been used for the purpose of verification.

The stress-strain curves for all types of brick masonry prisms have been found to be parabolic with very small curvature. The reason for small curvature is that the strain close to the failure could not be recorded. A quadratic mathematical model has been fitted for obtaining the equations of the stress-strain curves:

$$\sigma = a\,\varepsilon^2 + b\,\varepsilon \qquad \text{...(4.1)}$$

where, σ = compressive stress

ε = compressive strain

a, b = model parameters

The model parameters for different masonry types are given in Table 4.1. The corresponding values of R^2 are also given in the table. The modulus of elasticity of brick masonry obtained from Eq. (4.1) will be equal to the parameter *b*. which is reported in Table 4.1. The modulus of elasticity is taken as the slope of initial tangent.

Table 4.1 Model parameters for Stress-Strain Curves

Masonry type	Model parameters				R^2 value
	Actual stress		Nominal stress		
	a (MPa)	b (=E^*) (MPa)	a (MPa)	b (=E^*) (MPa)	
A	-1652.8	276.5	-1652.8	276.5	0.995
B	-1555.3	388.4	-1555.3	388.4	0.995
C	-1707.7	678.37	-1460.1	580.0	0.999
D	-2093.2	462.3	-1606.5	354.8	0.995

$^*E$ = modulus of elasticity of brick masonry

The value of modulus of elasticity of masonry Type A is less than other types because of large numbers of mortar joints in this type of masonry. The number of mortar joints in other type of masonry got reduced because of the use of bricks on edge. It can be been seen from Fig. 4.2 that the crushing strain for Type A masonry prisms is maximum among all types of masonry. Out of the three low cost masonry options, crushing strain is maximum for masonry Type B and minimum for masonry Type C, which shows that masonry type B is more ductile as compared to the other two.

4.2.2. Compressive Strength and Crushing Strain

The ultimate load, compressive strength and crushing strain for different types of brick masonry are given in Table 4.2 and the same have also been plotted in Figs. 4.4 and 4.5. The values of compressive strength and vertical load carrying capacity for full size bricks for different type of brick masonry are also reported in Table 4.2 and shown in the form of bar chart in Figs. 4.4 and 4.5. The compressive strength is the ultimate actual stress modified for the effect of slenderness ratio using Table 3.3 for making it comparable and the ultimate load is the load at failure. The compressive strength as well as the load carrying capacity of full size brick masonry is found to be about 40% of half size brick masonry for all types of masonry.

When masonry prism is loaded, the brick and mortar expand laterally because of the Poisson's effect. Since mortar expands more than the bricks, the bricks are subjected to lateral tension more than what they experience under uniaxial compression. Therefore, the strength of brick masonry is found to be less than the strength of the bricks and mortar.

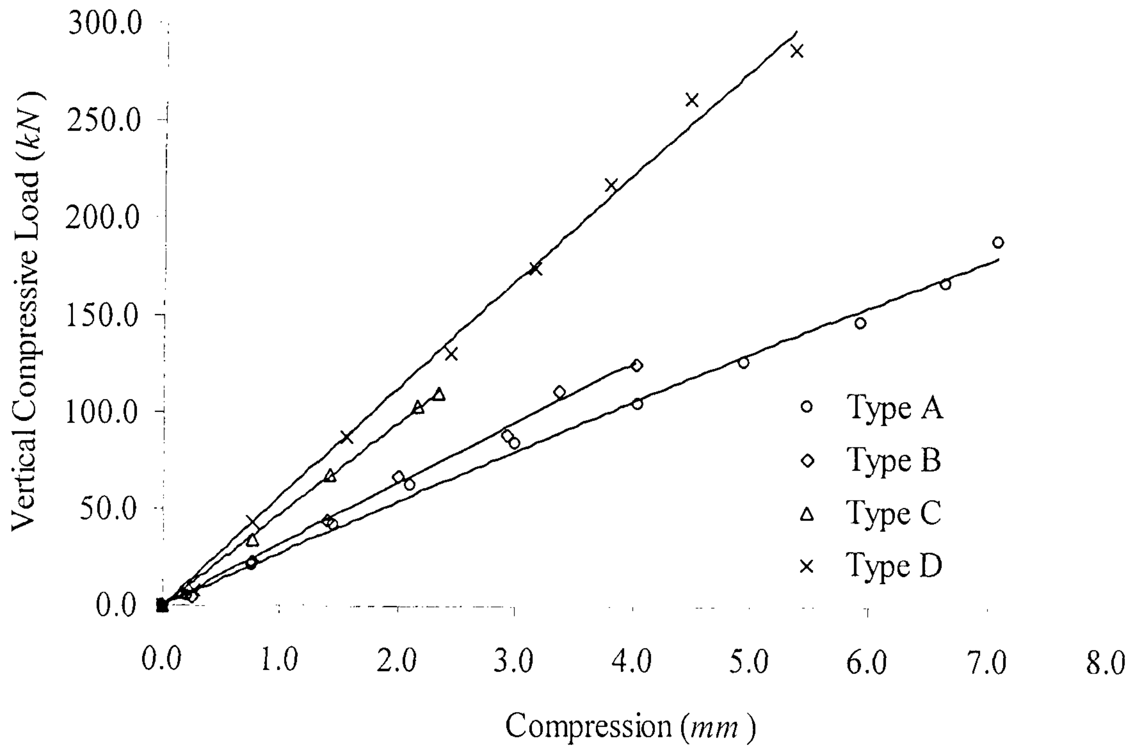


Fig. 4.1 Load Deformation Curves for Different Types of Brick Masonry Prisms

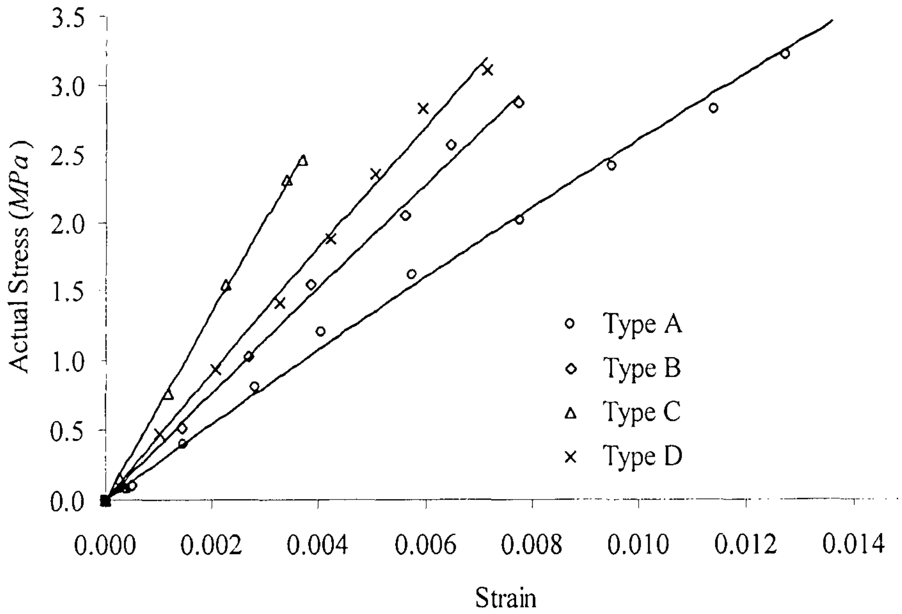


Fig. 4.2 Actual Stress-Strain Curves for Different Types of Brick Masonry Prisms

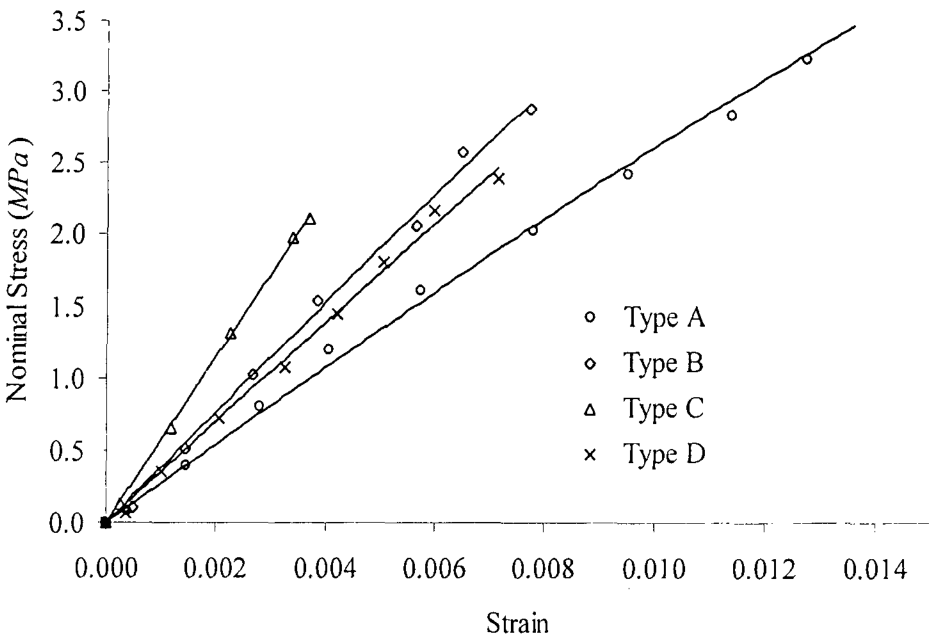


Fig. 4.3 Nominal Stress-Strain Curves for Different Types of Brick Masonry Prisms

The compressive strength of masonry Type D is maximum in comparison to other types of masonry. This is due to the fact that number of mortar joints for the same height in masonry type D is less than other masonry types, due to laying of all the courses of bricks on edge. But the vertical load carrying capacity of Type A masonry is more as compared to the masonry Type B, C, and D. It is because the load carrying capacity is based on the net area of the cross section which is maximum in the Type A brick masonry. The reduction in compressive strength of low cost masonry Type B and C as compared to Type D is 15% and 40% respectively. The reasons for the reduction in compressive strength of brick masonry Type B as compared to masonry Type A are:

- The vertical compression of courses containing bricks on edge will be less than the adjacent courses of bricks on bed due to lesser number of joints due to which the load distribution becomes non-uniform. It is due to this reason that the bricks on edge get crushed when tested in compression.
- The vertical mortar joint between the brick on edge and brick on bed is not properly filled as compared to the bricks on bed in masonry Type A due to which at ultimate load the brick on edge gets detached from bricks on bed.

A relationship used for the prediction of the compressive strength of brick masonry in terms of the mean strength of brick and mortar is as given below:

$$\sigma_{bm} = K_c\sqrt{\sigma_b \sigma_m} \qquad \ldots(4.2)$$

where, σ_{bm} = compressive strength of brick masonry

σ_b = average compressive strength of brick

= compressive strength on bed for Type A

= weighted mean of compressive strength on bed and on edge for Type B and C

= compressive strength on edge for Type D

σ_m = compressive strength of mortar

K_c = a coefficient for compressive strength of brick masonry

The standard deviation and coefficient of variation for brick masonry can be estimated from the individual values of bricks and mortar using the relationship given below and the same are given in Table 4.3.

$$s_{bm} = 0.5K_c\sqrt{\frac{\sigma_b}{\sigma_m}s_m^2 + \frac{\sigma_m}{\sigma_b}s_b^2} \qquad \ldots(4.3)$$

$$\delta_{bm} = 0.5\sqrt{\delta_b^2 + \delta_m^2} \qquad \ldots(4.4)$$

where, s_{bm}, s_b and s_m = standard deviation of strength of masonry, brick and mortar respectively;

δ_{bm}, δ_b and δ_m = coefficient of variation of strength of masonry, brick and mortar respectively.

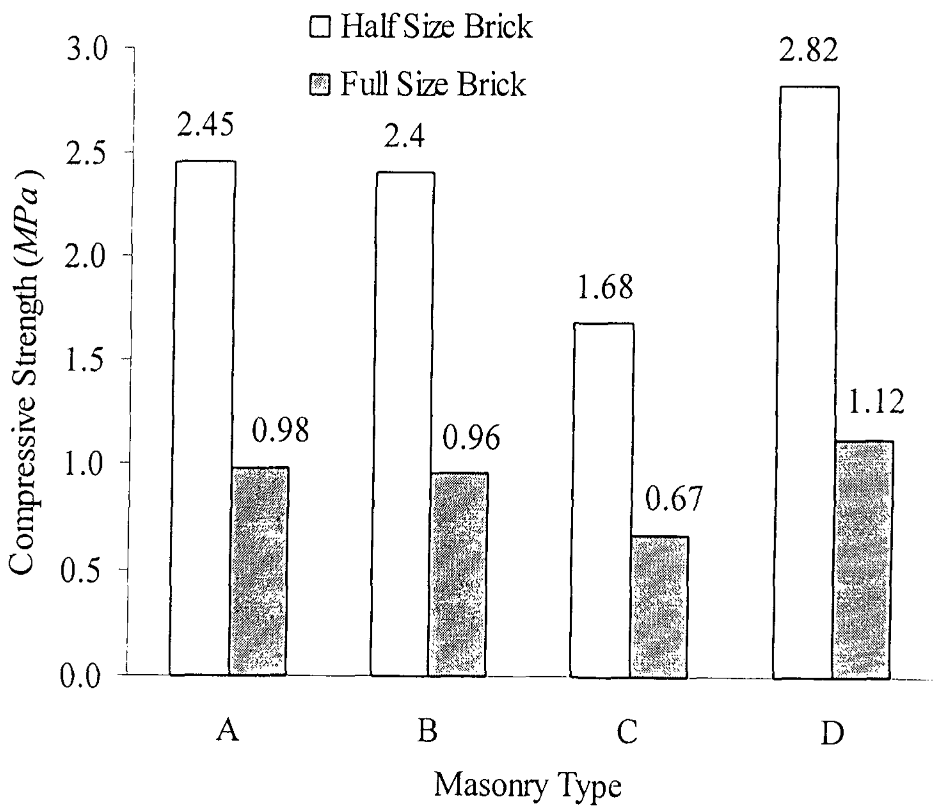


Fig. 4.4 Compressive Strength for Different Type of Masonry Prisms

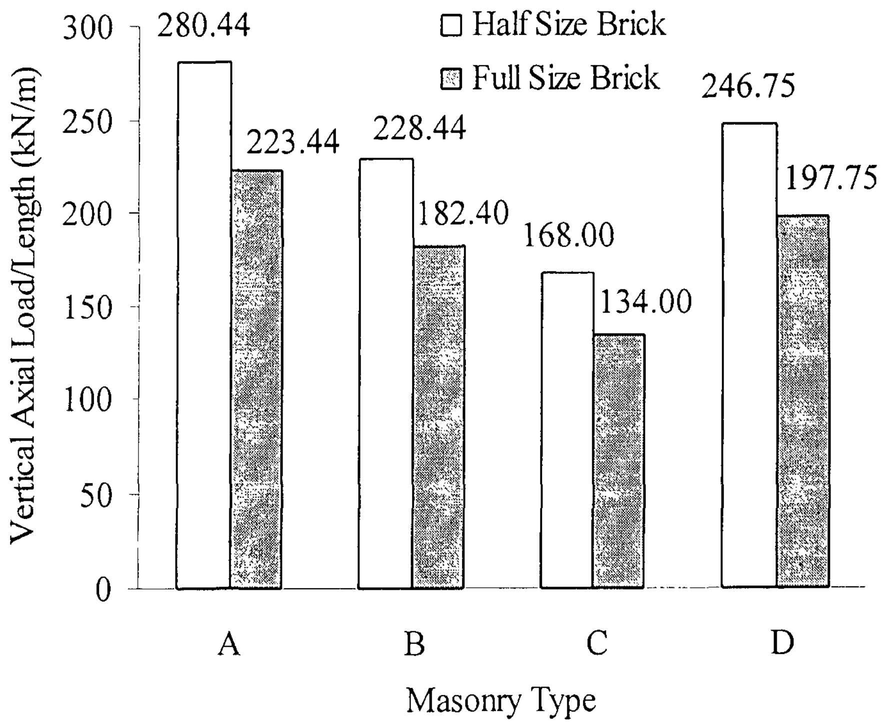


Fig. 4.5 Vertical Axial Ultimate Load per unit Length for Different Type of Masonry

Table 4.2 Compressive Strength, Crushing Strain and Maximum Vertical Axial Load per unit Length

Masonry type	Compressive strength (*MPa*)		Maximum vertical axial load per unit length (*kN/m*)		Crushing strain
	Half size brick	Full size brick	Half size brick	Full size brick	
A	2.46	0.98	280.44	223.44	1.22%
B	2.40	0.96	228.00	182.4	0.75%
C	1.68	0.67	168.00	134.00	0.41%
D	2.82	1.13	246.75	197.75	0.68%

Table 4.3 Standard Deviation and Coefficient of Variation for Compressive Strength of Prisms

Prism types	Actual thickness (mm)	Effective thickness (mm)	$\sigma_{bm}{}^{*}$ (MPa)	$K_c{}^{**}$	s_{bm} (MPa)	$\delta_{bm}\,(\%)$
A	228	228	2.46	0.31	0.24	9.89
B	190	190	2.40	0.32	0.29	12.82
C	228	195	1.68 (1.44)	0.29 (0.25)	0.23	10.85
D	228	175	2.82 (2.16)	0.40 (0.31)	0.44	15.80

*value within brackets is nominal stress, **value within brackets is corresponding to nominal stress.

4.2.3. Shear Strength

The shear strength and shear load carrying capacity per unit length of masonry found from experiments conducted on masonry prisms are plotted in Figs. 4.6 and 4.7. The pattern of variation of shear strength and shear load carrying capacity per unit length of masonry is obviously the same. The reduction in shear strength of masonry type B, C and D as compared to the conventional solid brick masonry i.e. type A is 20%, 52% and 21% respectively. A comparison of shear strength of masonry Type C and D, shows that the masonry Type D is stronger by 63% as compared to masonry Type C.

A relationship used for the prediction of shear strength of brick masonry in terms of the compressive strength of brick masonry is as given below:

$$\tau = K_{ss}\sqrt{\sigma_{bm}} \qquad \ldots(4.5)$$

where, τ = shear strength of masonry in MPa

σ_{bm} = compressive strength of masonry in MPa

K_{ss} = a coefficient, its value for different type of brick masonry prisms is given in Table 4.4.

Table 4.4 Coefficient, K_{ss}, for Shear Strength of Prisms

Prism types	Compressive strength, σ_{bm} (MPa)		K_{ss}	Shear strength, τ (MPa)	
	Half size brick	Full size brick		Half size brick	Full size brick
A	2.46	0.98	0.36	0.56	0.35
B	2.40	0.96	0.29	0.45	0.28
C	1.68	0.67	0.21	0.27	0.17
D	2.82	1.13	0.26	0.44	0.28

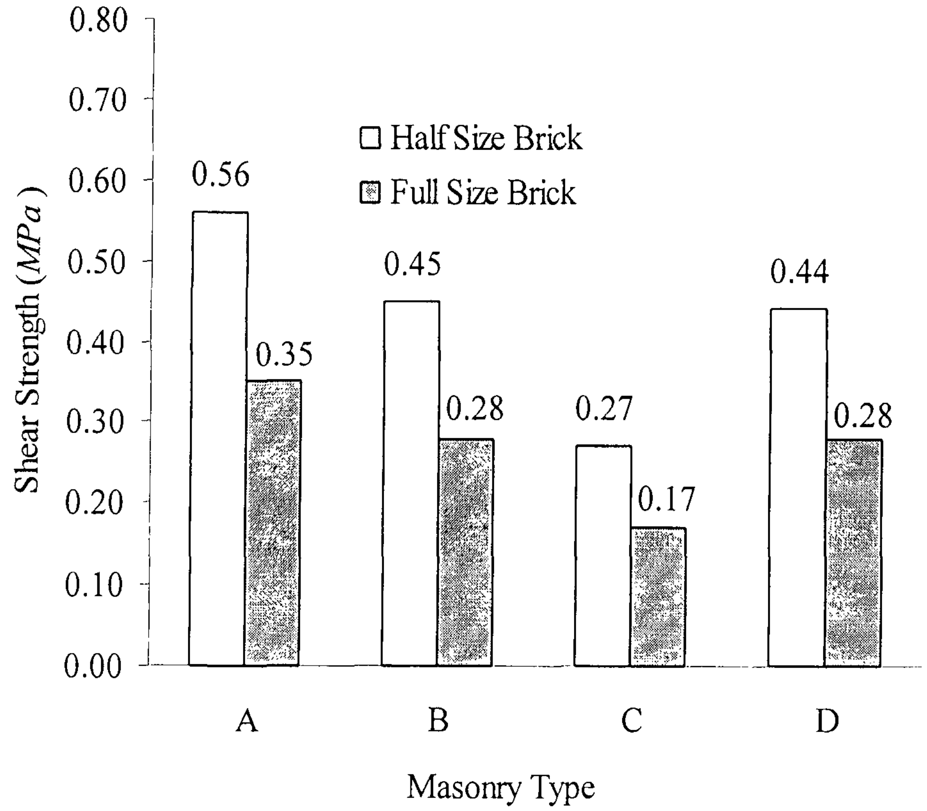


Fig. 4.6 Shear Strength of Different Type of Prisms

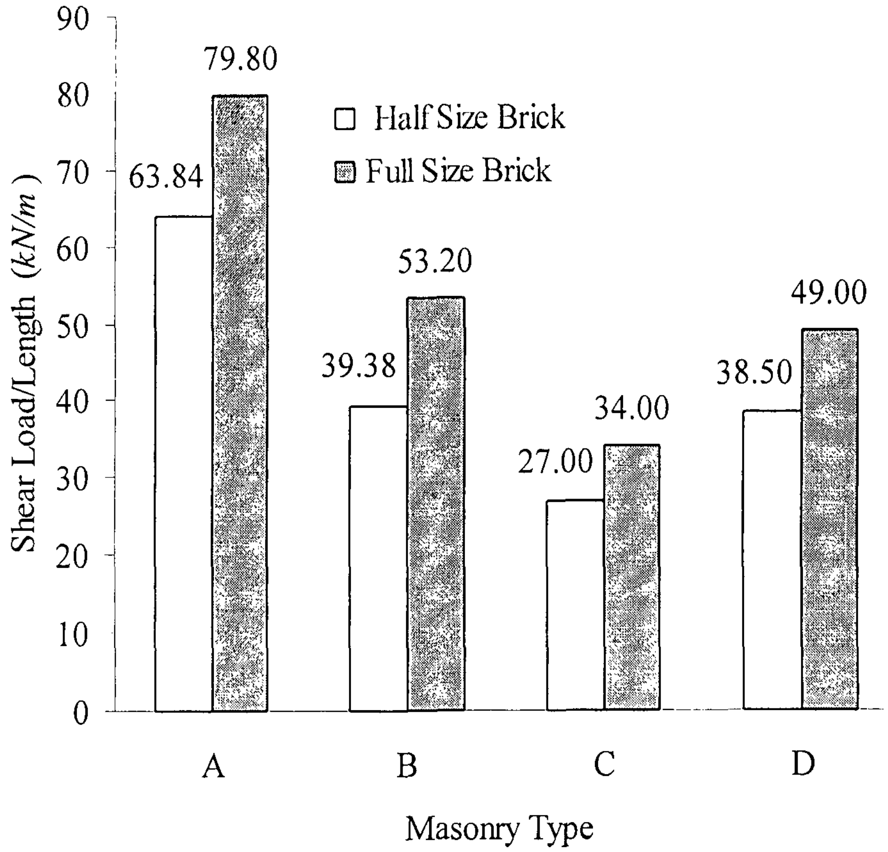


Fig. 4.7 Shear Load per unit Length for Different Type of Masonry

4.2.4. Mode of Failure

The failure patterns of different types of brick masonry prisms in compression and shear are shown in Figs. 4.8 and 4.9 respectively. It is observed from Fig. 4.8 that in most of the brick masonry prisms, cracks develop along the vertical mortar joints. This may be due to the following reasons:

i) Even if care is taken in laying of bricks, all bricks will not be evenly supported on the mortar bed due to which bricks are subjected to flexural and shear stresses along the vertical mortar joints. This may be responsible for the cracking along the vertical joints and these additional stresses will reduce the strength of prisms.

ii) Insufficient filling of mortar joints, varying thickness of bricks and joints give rise not only to the flexural stresses but also result in uneven distribution of external load. The stress concentration develops in the bricks, which may be considerably larger than the nominal average stress which may also be the cause of failure.

The prisms of masonry Type A failed by the development of vertical cracks along the vertical mortar joint which is due to the following reasons:

i) When masonry prism is loaded, the brick and mortar expand laterally because of the Poisson's effect. Since mortar expands more than the bricks, the bricks are subjected to lateral tension more than what they experience under uniaxial compression. Therefore the bricks fail in lateral tension.

ii) Another reason for the development of vertical crack along vertical mortar joint is that lateral tensile strength of the vertical mortar joint is less than the tensile strength of bricks.

The prisms of masonry Type B failed by the development of vertical cracks at mid length of bricks. In this type of brick masonry cracking is not along the vertical mortar joint because all of the vertical mortar joints are not in same vertical line. The cracking of bricks at mid length is due to lateral tension getting developed in the bricks because of the lateral expansion, due to the Poisson's effect.

In the prisms of masonry Type C, the failure is by the development of vertical cracks in the bricks on the bed. This is because of the lateral tension getting developed in these bricks.

In the masonry Type D, vertical cracks have developed in the header bricks which are due to the development of lateral tension because of the Poisson's effect. Vertical cracks have also developed along the vertical mortar joint which is due to the development of lateral tension in bricks in the transverse direction.

All the masonry prisms tested in shear failed by sliding along the horizontal mortar bed which is the weakest plane in shear.

4.3. LATERAL STRENGTH OF WALL PANELS

The experimental results of Type A, B and D brick masonry wall panels tested under lateral loads are given in Table 4.5 and their failure pattern is shown in Fig. 4.10. The masonry Type C was not considered because of its very low compressive strength as observed in the previous section. The deformation of diagonals of the wall panels given in the table is the average of the two Demac gauges readings along a diagonal. The ultimate shear strength of brick masonry is calculated on the basis of net cross-sectional area. Though the ultimate lateral load for masonry Type B and D is 29% and 38% less than that of masonry Type A respectively, whereas, shear strength of masonry Type B and D is 13% and 6.5% less than that of masonry Type A respectively. The reduction in shear strength for masonry Type B is due to the non-uniform distribution of load on mortar bed as discussed earlier in Sec. 4.2.2. The ultimate shear strain is maximum for masonry Type D which shows that this type of masonry is more ductile under lateral load as compared to other types of masonry.

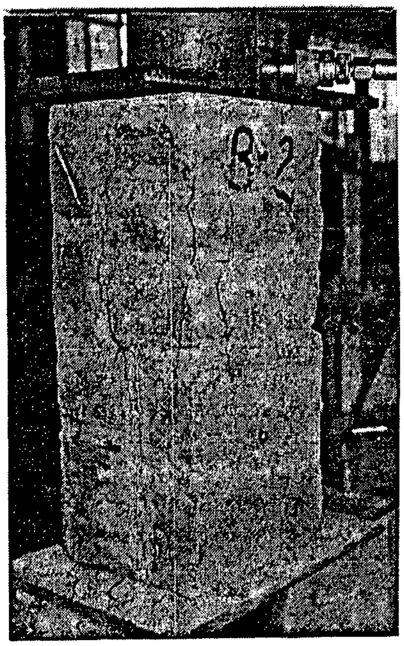
The wall panels of masonry Type A and B failed due to sliding at the base along the horizontal mortar bed (Figs. 4.10 (a) and (b)) which is because of the masonry being stronger along the diagonals. Whereas the wall panel of masonry Type D failed in diagonal tension by the development of cracks along the diagonal passing through horizontal and vertical mortar joints (Fig. 4.10 (c)) because this type of masonry is relatively weaker in tension along its diagonals. The reduction in tensile strength along the diagonal is because of the vertical mortar joint being relatively weaker because the height of vertical mortar joint in masonry Type D is more as compared to masonry Type A. The cracks in the wall panels of all types of brick masonry got developed only at the failure load and there was no any visible crack at lesser magnitude of load.

The lateral load is plotted against horizontal deflection at the top and mid height of the wall in Figs. 4.11 and 4.12 respectively. The initial slope of the load deflection curve for masonry Type A is steeper as compared to other masonry types. The initial slope of the load deflection curve for masonry Type D is steeper than masonry Type B. The deflection at failure for masonry Type D is much larger than other types of masonry thus showing large amount of ductility against lateral loads.

The diagonal compression and extension at failure in walls of masonry Type A and B are almost same and their magnitude is quite small. Whereas, in the wall of masonry Type D, diagonal extension at failure is large due to development of diagonal tension cracks. The magnitude of diagonal compression in masonry Type D is quite small.

Table 4.5 Ultimate Lateral Load and Shear Strength with Mode of Failure for Different Wall Panels

Masonry types	Ultimate lateral load (*kN*)	Ultimate shear strength (*MPa*)	Maximum deflection at top of wall (*mm*)	Average elongation of diagonal (*mm*)	Average contraction of diagonal (*mm*)	Mode of failure
A	41.0	0.15	1.77	0.055	0.060	Sliding at base
B	29.0	0.13	3.13	0.030	0.025	Sliding at base
D	25.5	0.14	3.22	5.000	0.010	Diagonal tension

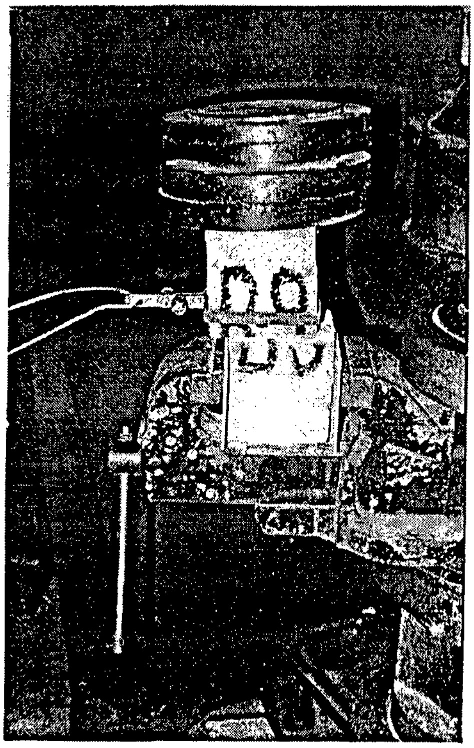


(a) Masonry Type B

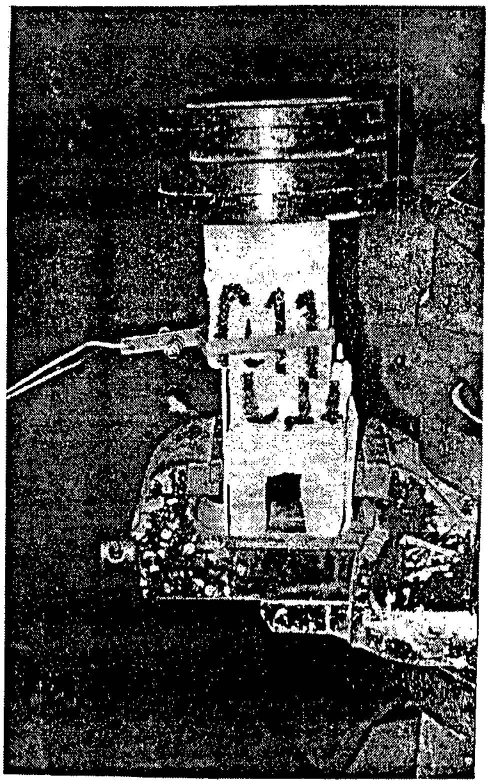
(b) Masonry Type C

(c) Masonry Type D

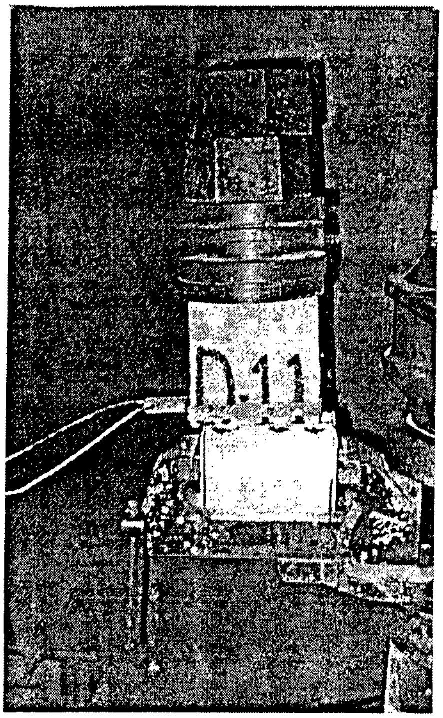
Fig. 4.8 Failure Patterns of Different Type of Prisms in Compression



(a) Masonry Type B

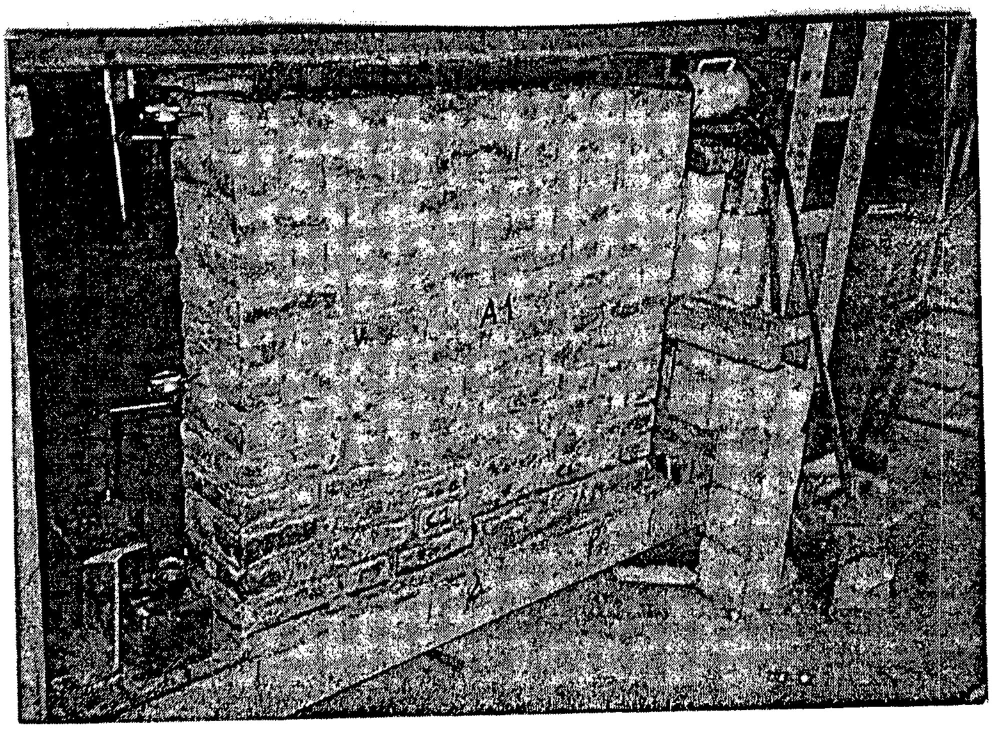


(b) Masonry Type C

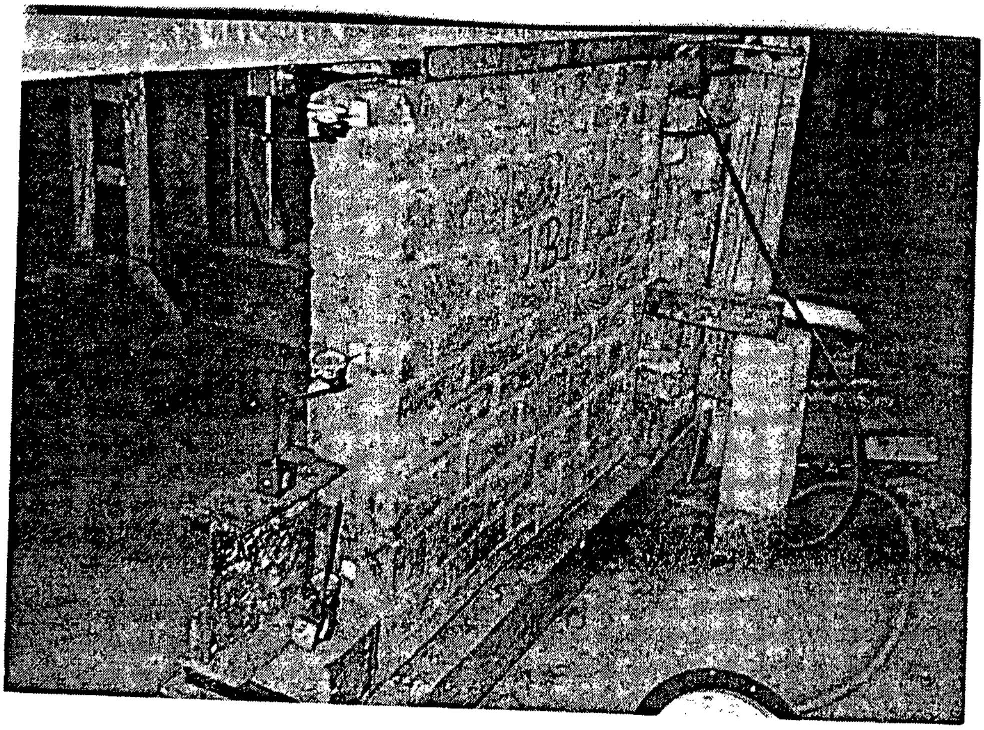


(c) Masonry Type D

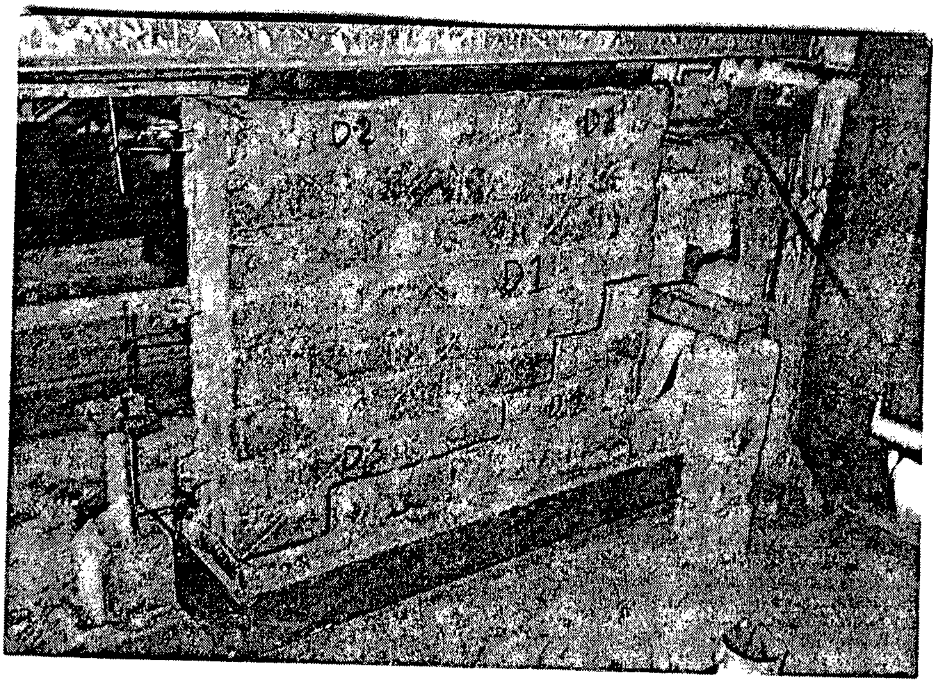
Fig. 4.9 Failure Pattern of Different Type of Prisms in Shear



(a) Type A Brick Masonry Wall Panel



(b) Type B Brick Masonry Wall Panel



(c) Type D Brick Masonry Wall Panel

Fig. 4.10 Failure Pattern of Different Types of Masonry Wall Panels under Static Lateral Load

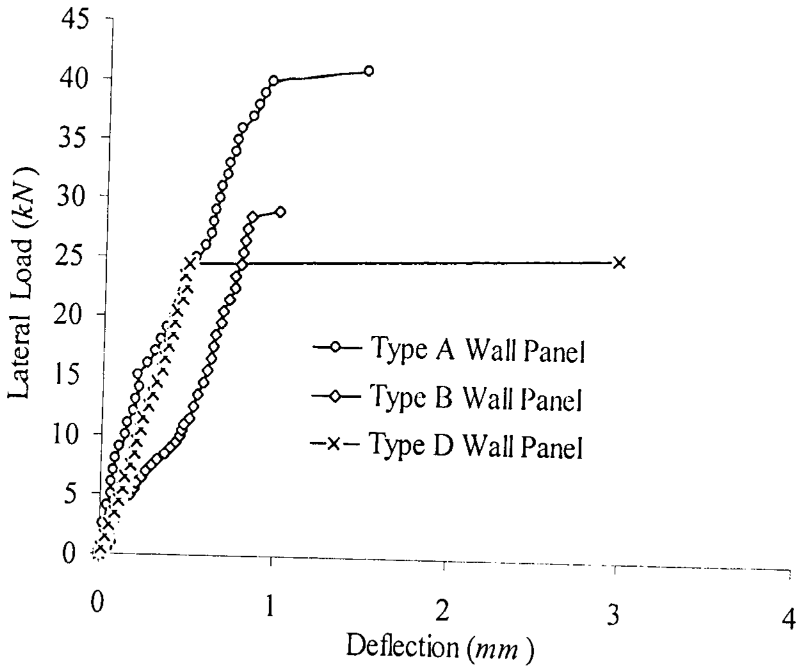


Fig. 4.11 Load Deflection Curves for Different Wall Panels at Top

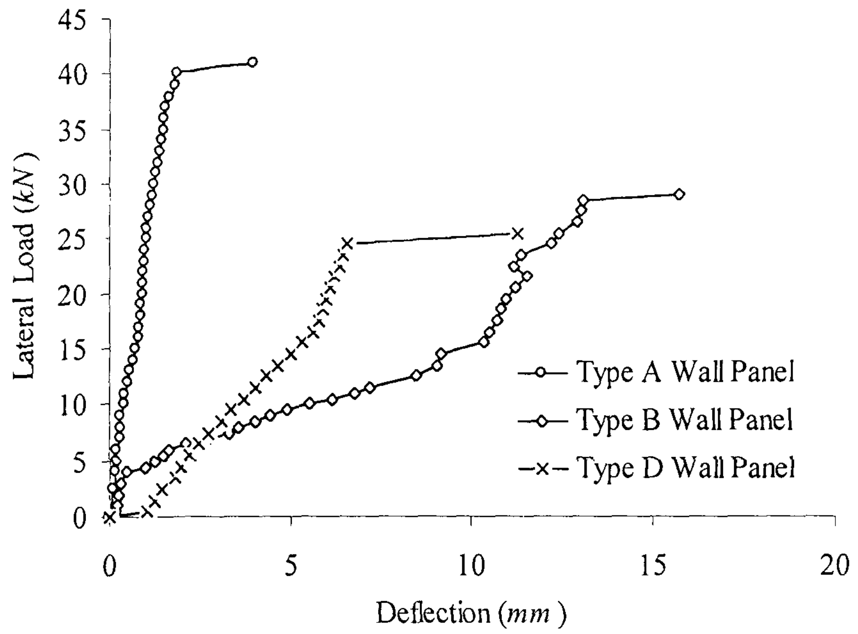


Fig. 4.12 Load Deflection Curves for Different Wall Panels at Mid Height

4.3.1. Finite Element Analysis

A simplified Finite element analysis was carried out with the help of STAAD-Pro package for studying the effect of the thickness of mortar joint. The thickness of mortar joint was varied by taking it as 6, 8, 10, 12 and 14 *mm*. In the process of discretization, bricks and mortar were represented by separate four node linear plate elements. The properties of materials were taken from the experimental test results reported in Chapter 3. The assumptions made in the analysis are as follows:

- The brick masonry is considered as linear elastic material.
- Perfect bond is assumed between bricks and mortar.
- The variation in stresses across the thickness of wall is ignored.

The magnitude of maximum shear stress and maximum absolute stress are given in Table 4.6. It is found from the table that the maximum shear stress and maximum absolute stress increases with increase in the thickness of mortar joints thus making the masonry weaker. The thickness of mortar should, therefore, be kept small. It should be minimum required for leveling the horizontal mortar bed.

Table 4.6 Maximum Shear and Absolute Stress in Different Wall Panels

Thickness of mortar joint (*mm*)	Maximum shear stress (*MPa*)	Maximum absolute stress (*MPa*)
6	0.0192	0.0999
8	0.0196	0.1020
10	0.0198	0.1027
12	0.0202	0.1055
14	0.0205	0.1068

4.3.2. Lateral Stiffness of Wall

Considering a wall of height H, length L_w and thickness t_w as shown in Figs. 4.13 and 4.14. Two different cases of openings are considered – one in which there is a door and a ventilator (Fig. 4.13) and second in which there is a window (Fig. 4.14). Some other cases of openings are also covered such as wall without opening and wall with a door opening without ventilator. The openings are assumed to be centrally placed. The sizes of openings are: door ($L_1 \times H_1$), ventilator ($L_2 \times H_2$) and window ($L_3 \times H_3$).

The base of the wall is considered as fixed because of its connection with massive footing. The top of the wall is also considered as fixed but free for lateral displacement because of its connection with the slab at the top. The wall is given unit lateral displacement at the top by applying a horizontal force for the purpose of calculating the lateral stiffness of wall. The total strain energy of the wall, U, for this mode of deformation can be calculated from the summation of the strain energy due to bending moment and shear:

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$$U = \int_{x=0}^{L_w} \frac{M_x^2}{2EI} dx + \int_{x=0}^{L_w} \int_{y=y_t}^{y_c} \frac{q^2}{2G} zdydx \qquad \ldots(4.6)$$

where, M_x = bending moment at a section distant x from base

q = shear stress intensity at a section

z = width of the fibre at a distance y from neutral axis

I = second moment of area of the section of wall about neutral axis

E = modulus of elasticity for brick masonry

G = shear modulus for brick masonry

y = distance of fibre under consideration from neutral axis

y_t = distance of the fibre in tension zone under consideration from the neutral axis

y_c = distance of the fibre in compression zone under consideration from the neutral axis

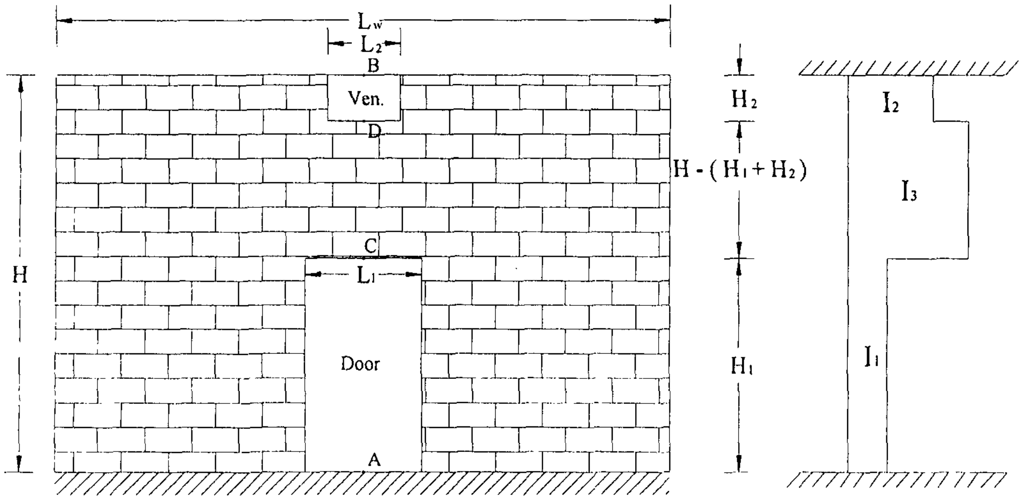
The magnitude of shear stress, q, for different segments of the wall is calculated from the relation:

$$q = \frac{F}{IZ}\left(A\bar{y}\right) \qquad \ldots(4.7)$$

which is given in Table 4.7.

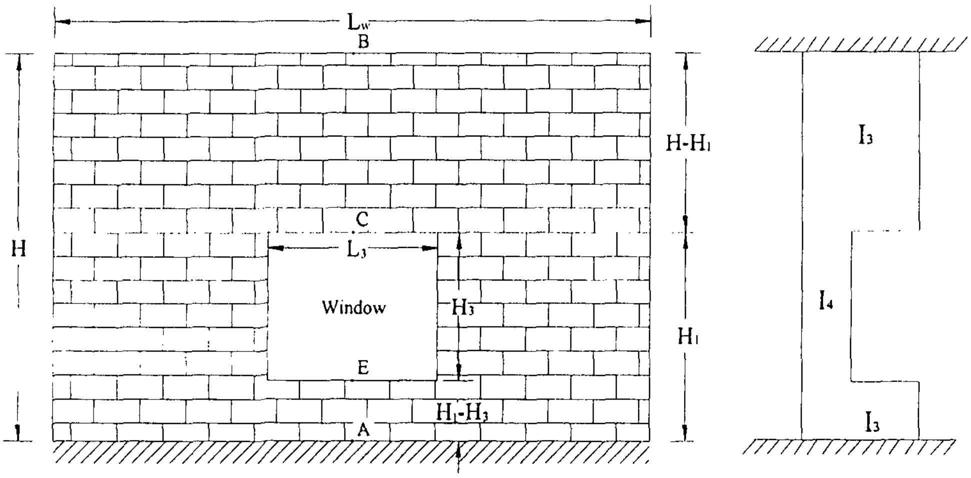
where, F = shear force at a section distant x from base

$A\bar{y}$ = moment of area of the portion which is between the fibre under consideration and the extreme of fibre



(a) Front Elevation of the Wall (b) Variation of Second Moment of Area

Fig. 4.13 Brick Masonry Wall with Door and Ventilator



(a) Front Elevation of the Wall (b) Variation of Second Moment of Area

Fig. 4.14 Brick Masonry Wall with Window

Table 4.7 Equations for Shear Stress for Different Types of Walls

Type of walls	Wall segment	Shear stress, q	Limit of y
Wall having door and ventilator as shown in Fig. 4.13	AC	$\frac{F}{2I_1}\left(\frac{L_w{}^2}{4}-y^2\right)$	$\pm\frac{L_1}{2}$ to $\pm\frac{Lw}{2}$
	CD	$\frac{F}{2I_3}\left(\frac{L_w{}^2}{4}-y^2\right)$	0 to $\pm\frac{L_w}{2}$
	DB	$\frac{F}{2I_2}\left(\frac{L_w{}^2}{4}-y^2\right)$	$\pm\frac{L_2}{2}$ to $\pm\frac{Lw}{2}$
Wall having window as shown in Fig. 4.14	AE and CB	$\frac{F}{2I_3}\left(\frac{L_w{}^2}{4}-y^2\right)$	0 to $\pm\frac{L_w}{2}$
	EC	$\frac{F}{2I_4}\left(\frac{L_w{}^2}{4}-y^2\right)$	$\pm\frac{L_3}{2}$ to $\pm\frac{Lw}{2}$

*I_1, I_2, I_3 and I_4 are second moment of area of the wall for the portion having door opening, ventilator opening, without opening and window opening respectively about neutral axis.

Applying a horizontal fictitious load, P, at the top of the wall, the bending moment and shear force at a section distant x from the base of the wall is given by:

$$M_x = (V - P)x + M \qquad \ldots(4.8)$$

$$F = (V - P) \qquad \ldots(4.9)$$

where, M is the fixed end moment. Minimization of the strain energy gives:

$$\frac{\partial U}{\partial M_x} = 0 \text{ and } \frac{\partial U}{\partial P} - 1 = 0 \qquad \ldots(4.10)$$

From the above equations, we get the following two linear simultaneous equations for each case of wall opening considered in the study.

Wall with Door and Ventilator Opening (Fig. 4.13):

$$\begin{aligned}&\frac{1}{EI_1}\left[\frac{VH_1^3}{3}+\frac{MH_1^2}{2}\right]\\&+\frac{1}{EI_3}\left[\frac{V\left\{(H-H_2)^3-H_1^3\right\}}{3}+\frac{M\left\{(H-H_2)^2-H_1^2\right\}}{2}\right]\\&+\frac{1}{EI_2}\left[\frac{V\left\{H^3-(H-H_2)^3\right\}}{3}+\frac{M\left\{H^2-(H-H_2)^2\right\}}{2}\right]\\&+\frac{Vt_w}{120G}\left[\left(L_w^5-L_1^5\right)\frac{H_1}{I_1^2}+\frac{L_w^5}{4}\frac{(H-H_1-H_2)}{I_3^2}+\left(L_w^5-L_2^5\right)\frac{H_2}{I_2^2}\right]-1=0\end{aligned}\qquad\ldots(4.11)$$

and

$$\begin{aligned}&\frac{1}{EI_1}\left[\frac{VH_1^2}{2}+MH_1\right]\\&+\frac{1}{EI_3}\left[\frac{V\left\{(H-H_2)^2-H_1^2\right\}}{2}+M(H-H_1-H_2)\right]\\&+\frac{1}{EI_2}\left[\frac{V\left\{H^2-(H-H_2)^2\right\}}{2}+MH_2\right]=0\end{aligned}\qquad\ldots(4.12)$$

Wall with Window Opening (Fig. 4.14):

$$\begin{aligned}&\frac{1}{EI_3}\left[\frac{V\left\{(H_1-H_3)^3+H^3-H_1^3\right\}}{3}+\frac{M\left\{(H_1-H_3)^2+H^2-H_1^2\right\}}{2}\right]\\&+\frac{1}{EI_4}\left[\frac{V\left\{H_1^3-(H_1-H_3)^3\right\}}{3}+\frac{M\left\{H_1^2-(H_1-H_3)^2\right\}}{2}\right]\\&+\frac{Vt_w}{120G}\left[\frac{L_w^5}{4}\frac{(H-H_3)}{I_3^2}+\left(L_w^5-L_3^5\right)\frac{H_3}{I_4^2}\right]-1=0\end{aligned}\qquad\ldots(4.13)$$

and

$$\frac{1}{EI_3}\left[\frac{V\left\{(H_1 - H_3)^2 + H^2 - H_1^2\right\}}{2} + M(H - H_3)\right]$$
$$+\frac{1}{EI_4}\left[\frac{V\left\{H_1^2 - (H_1 - H_3)^2\right\}}{2} + MH_3\right] = 0 \qquad \ldots(4.14)$$

where, t_w = thickness of wall

The above equations may be solved to get the value of M and V hence the lateral stiffness of the wall for the two cases of wall openings.

The lateral stiffness of walls of different types of masonry tested experimentally has been calculated from Eqs. (4.11) to (4.14) and the values are given in Table 4.8. The thickness of wall for different types of masonry is taken as the effective thickness of wall. The lateral stiffness of wall of masonry Type A is less due to lesser value of modulus of elasticity as compared to other masonry types. It is observed from the table that the influence of shear deformation on the lateral stiffness of different types of masonry wall is 50.90%.

The lateral stiffness of a typical wall, 3 *m* high, 3 *m* long and 0.228 *m* thick of masonry Type A, has been calculated for different cases of door, window and ventilator openings and the same are given in Table 4.9. The size of doors, window and ventilator are given in the table.

It is observed from Table 4.9 that the consideration of ventilator just above the door increases the stiffness of wall by merely 0.63% with shear deformation as compared to the case when the ventilator is at the top of the wall. Two widths of window opening, 40% and 60% of length of wall, have been considered in Table 4.9. A comparison of stiffness shows that 50% increase in the width of window reduces the stiffness by 18.28% with shear deformation. The presence of opening such as a window of size 1.8×1.95 m reduces the stiffness of wall without opening by 58.58% with shear deformation. The consideration of shear deformation reduces the stiffness of wall by 42% to 74% for the cases considered in the table.

Table 4.8 Lateral Stiffness of Different Types of Brick Masonry Walls without Opening

Type of masonry	Lateral stiffness of wall without opening (*kN/mm*)	
	Without shear	With shear
A	108.74	53.39
B	127.39	62.54
D	139.71	68.59

Table 4.9 Lateral Stiffness of Brick Masonry Wall with Different Cases of Openings

S. No.	Particulars	Lateral stiffness of wall (*kN/mm*)	
		Without shear	With shear
1.	Wall without opening	63.0	36.7
2.	Wall with door and ventilator (D = 1.2×2.1 *m*; V = 1.2×0.45 *m*); ventilator at top of wall	59.5	16.0
3.	Wall with door and ventilator (D = 1.2×2.1 *m*; V = 1.2×0.45 *m*); ventilator just above door	60.3	16.1
4.	Wall with window (W = 1.2×1.95 *m*)	61.9	18.6
5.	Wall with window (W = 1.8×1.95 *m*)	58.5	15.2

4.4. STATIC RESPONSE OF BUILDING MODELS

The notations used for different walls of the model inscribed on them are as follows:

<u>For Models HM-1 (Masonry Type A) and HM-3 (Masonry Type D):</u>

F-1 is the wall in which door is installed;

F-2 is the wall on which static lateral load at the top is applied;

F-3 is the wall in which window is installed; and

F-4 is the wall opposite to F-2, deflection was recorded on this wall.

For Model HM-2 (Masonry Type B):

F-1 is the wall in which window is installed;

F-2 is the wall on which static lateral load at the top is applied;

F-3 is the wall in which door is installed; and

F-4 is the wall opposite to F-2, deflection was recorded on this wall

4.4.1. Crack Pattern

The cracks observed in different walls of various building models are shown in Figs. 4.15 to 4.17. The crack Identification marks viz. C1, C2 and C3 are used for cracks developed at increasing magnitude of lateral load. The crack identification marks are written on the walls of the building models and are shown in Figs. 4.15 to 4.17. The propagation of cracks in different walls of the building models at increasing magnitude of lateral load, are given in Table 4.10. The cracks developed in the models are mainly diagonal, whereas, some of the cracks are horizontal and close to the slab. All the cracks observed in different building models are along the mortar joints.

The observed pattern of diagonal cracks in the shear walls are due to the combined effect of elongation of far-diagonal, contraction of near-diagonal, fixity at the base and brittleness of the masonry. Whereas, the cracks near the edges may be due to some invisible eccentricity and stress concentration. The pattern of dominant horizontal cracks in the cross walls are due to combined effect of lateral deflection of building in the direction of load and brittleness of brick masonry which reduces the shear strength of brick masonry. The different types of crack patterns observed on different faces of building models provide strengthening criteria of such buildings against destructive forces that are likely to act on buildings during service condition.

The crack patterns observed in wall panels tested independently under lateral load and the walls of building models are quite different. This is because of the box action and the presence of openings in the walls of the building models.

4.4.2. Deformation and Deflection of Building Models

(i) *Walls Parallel to the Direction of Load*

The variation of mean elongation and contraction of the diagonals with lateral load is plotted in Figs. 4.18 to 4.19. The magnitude of elongation is more than the contraction of diagonal which is due to the fact that the tensile strength of brick masonry is less than its compressive strength.

(ii) *Cross Walls*

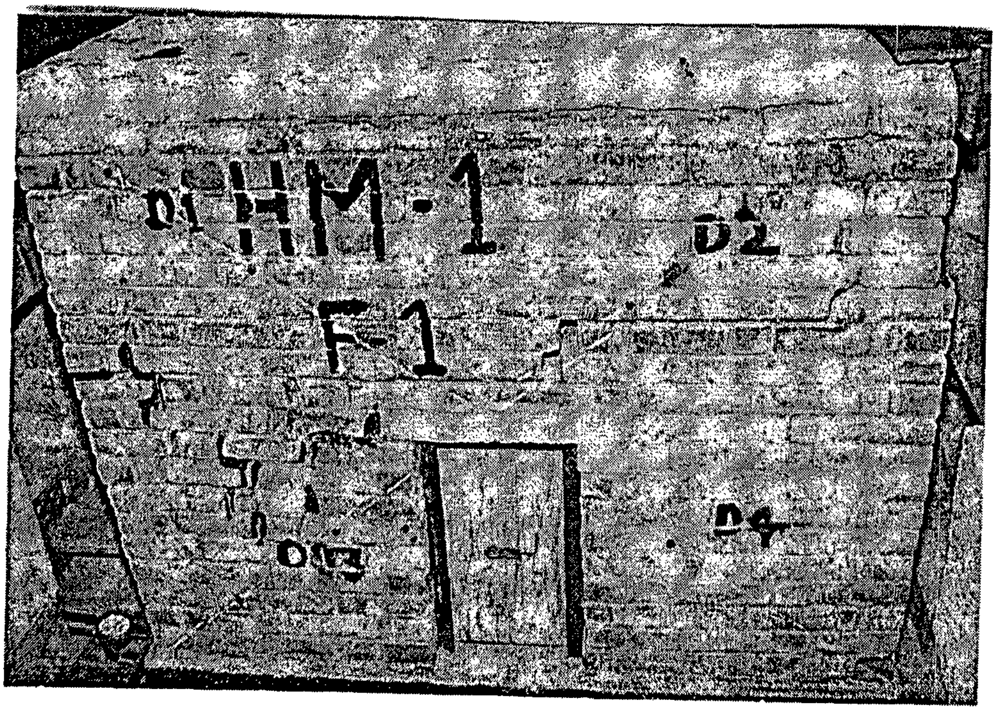
The deflection of the base of the building model recorded by the dial gauges placed on the wall opposite to the loaded face wall is zero for all the building models which shows that the base remained fixed during the application of load. The maximum deflection at the top of different building models recorded by dial gauge is given in Table 4.11. The variation of deflection at top with lateral load is shown in Fig. 4.20 and the variation of shear load per unit length with shear stain is shown in Fig. 4.21.

It is observed from Fig. 4.21 that building model HM-1 carries maximum lateral load with moderate peak deflection, while HM-3 carries minimum lateral load with maximum deflection at failure. This shows that the building model HM-3 constructed with masonry Type D masonry is more ductile as compared to other types of brick masonry considered in the study.

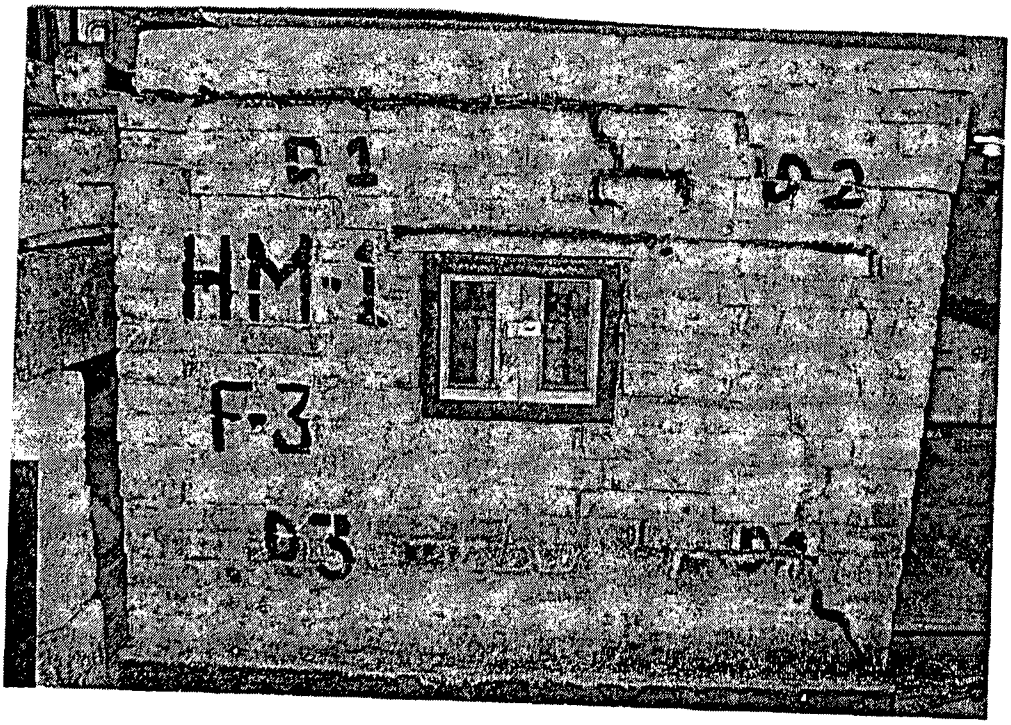
The experimental observations of deflection on the opposite cross wall (wall F-4) of different building models show that the deflection at the two corners at the top are not the same which may be due to small rotation of building resulting from the unsymmetrical building configuration.

Table 4.10 Crack Propagation Observed in Different Building Models Tested under Static Lateral Load

S. No.	Lateral load (*kN*)	Crack Propagation				Remark
		Wall with door	Wall with window	Wall on which load was applied	Wall on which deflection was recorded	
Model HM-1 (Masonry Type A)						
1	0.00					
2	0.50					
3	1.00					
4	1.50					
5	2.0	C1	C1	C1		
6	2.5	C2	C2	C2	C1	Ultimate Load
Model HM-2 (Masonry Type B)						
1	0.00					
2	0.50					
3	1.00	C1		C1		
4	1.50	C2	C1	C2		
5	2.00	C3	C2	C3	C1	Ultimate Load
Model HM-3 (Masonry Type D)						
1	0.00					
2	0.50					
3	1.00	C1	C1	C1	C1	
4	1.50	C2	C2	C2	C2	Ultimate Load

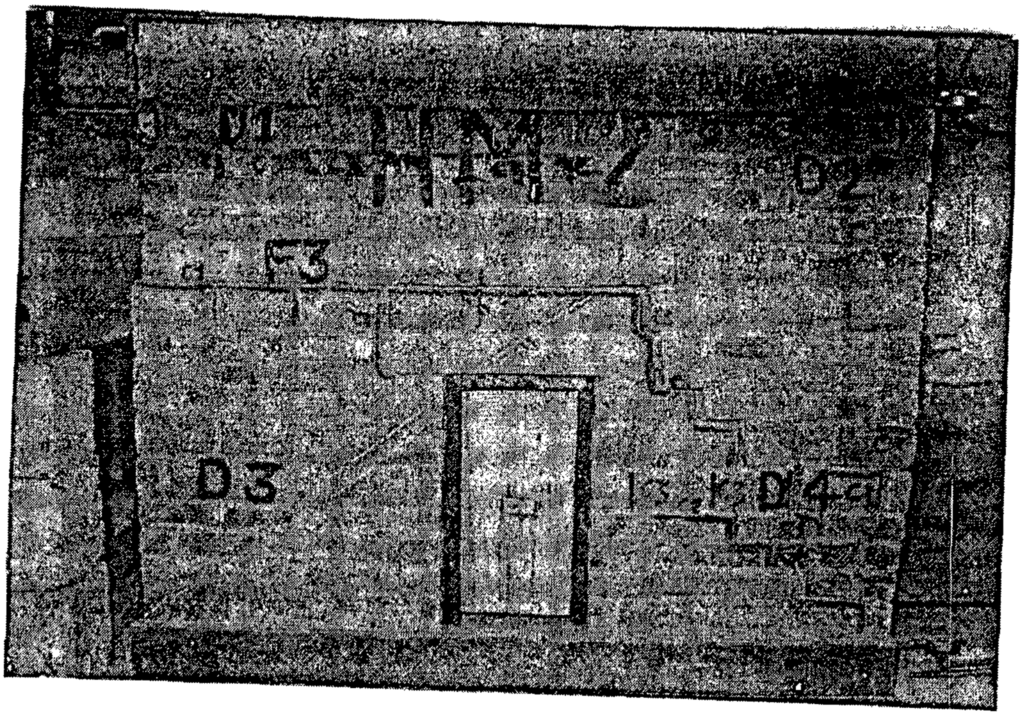


(a) Door Side Wall

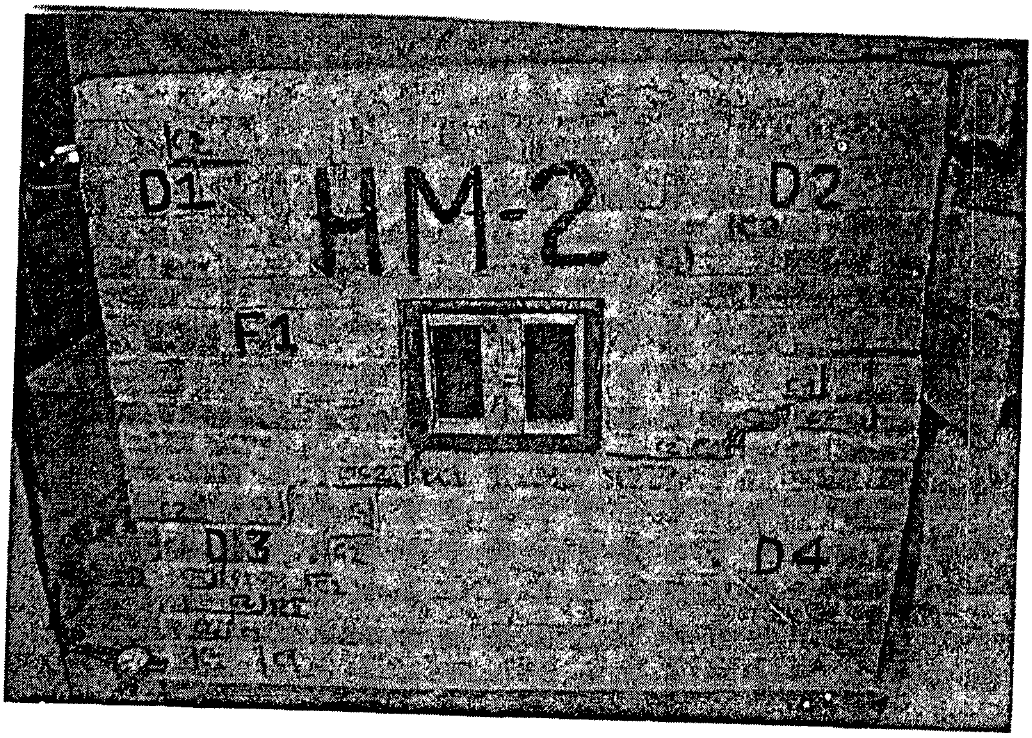


(b) Window Side Wall

Fig. 4.15 Crack Patterns in Building Model HM-1 (Masonry Type A) Tested under Static Lateral Load



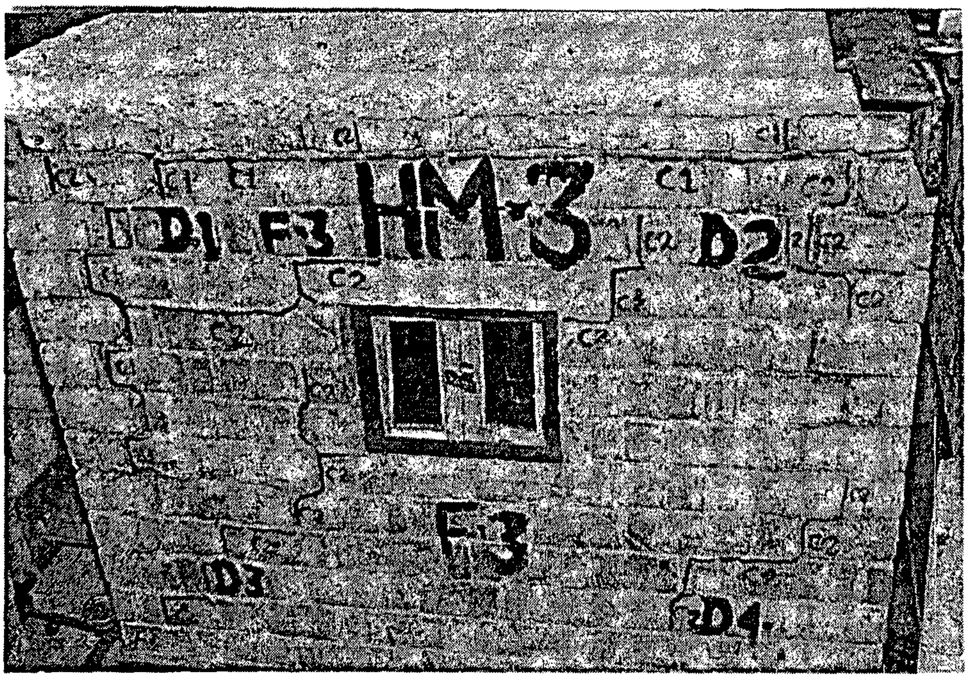
(a) Door Side Wall



(b) Window Side Wall

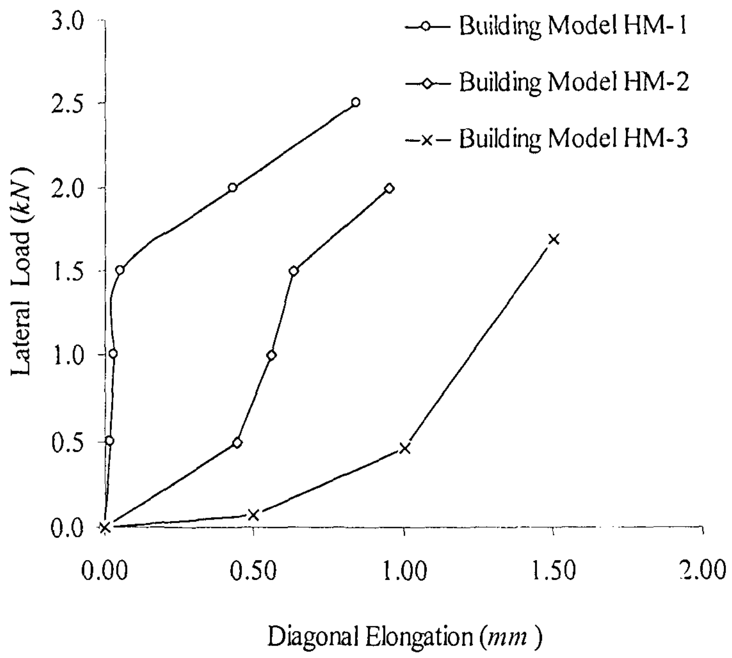
Fig. 4.16 Crack Patterns in Building Model HM-2 (Masonry Type B) Tested under Static Lateral Load

(a) Door Side Wall

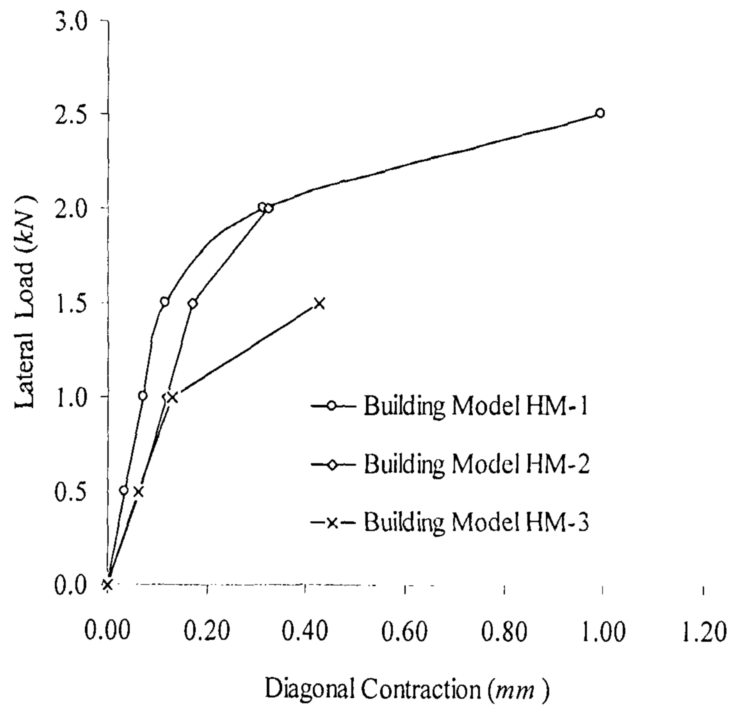


(b) Window Side Wall

Fig. 4.17 Crack Patterns in Building Model HM-3 (Masonry Type D) Tested under Static Lateral Load

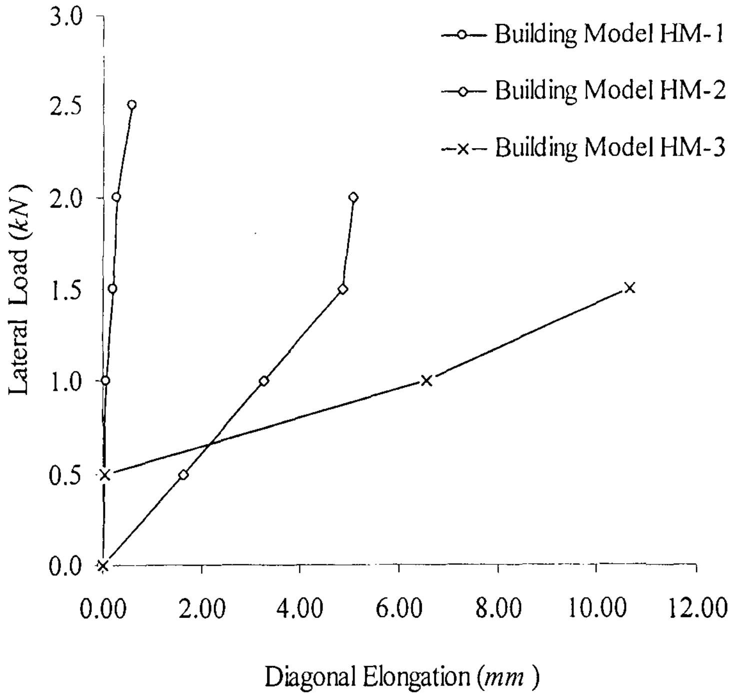


(a) Diagonal Getting Elongated

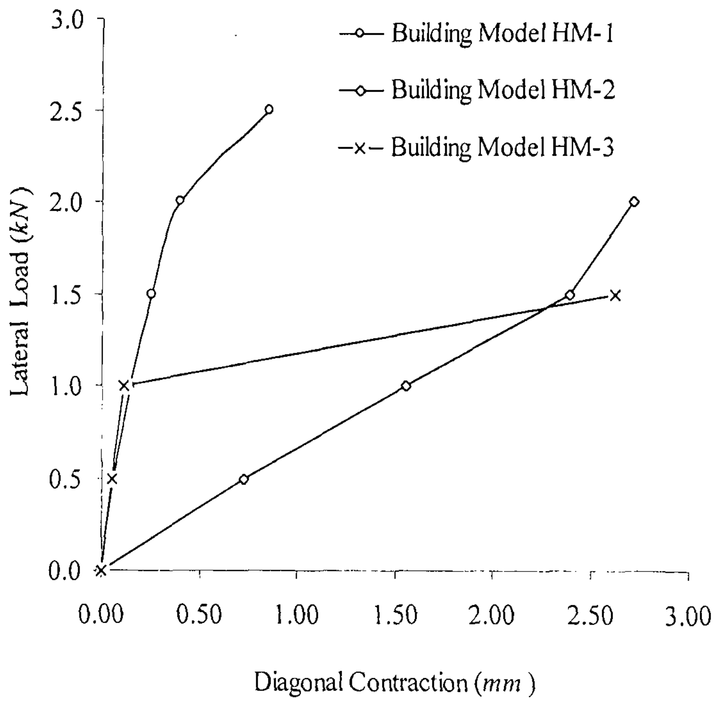


(b) Diagonal Getting Compressed

Fig. 4.18 Load Deformation Curves for Door Side Wall of Different Building Models



(a) Diagonal Getting Elongated



(b) Diagonal Getting Compressed

Fig. 4.19 Load Deformation Curve for Window Side Wall of Different Building Models

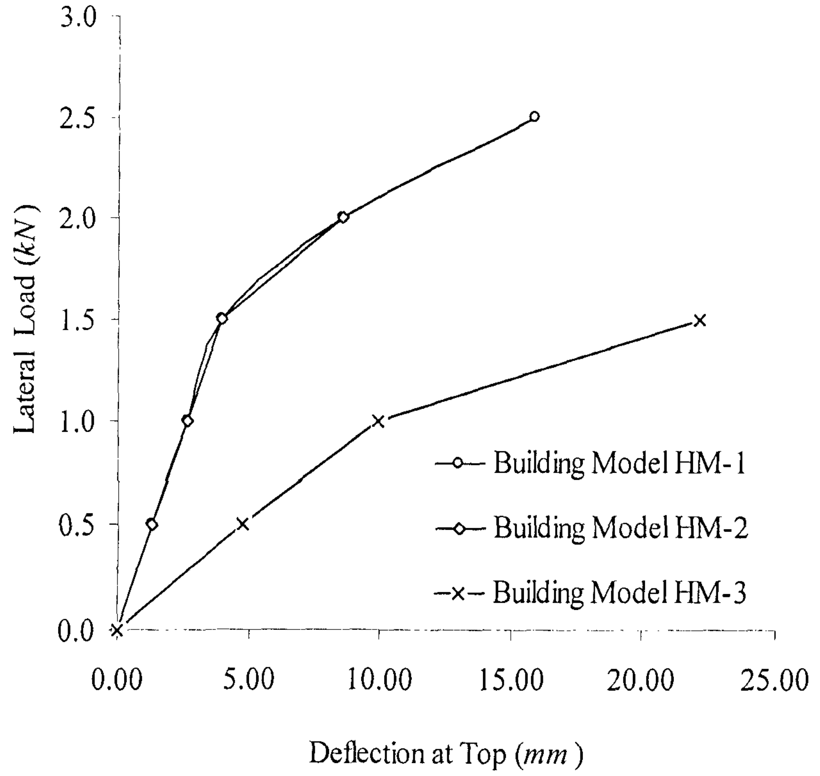


Fig. 4.20 Load Deflection Curves for Top of Different Building Models

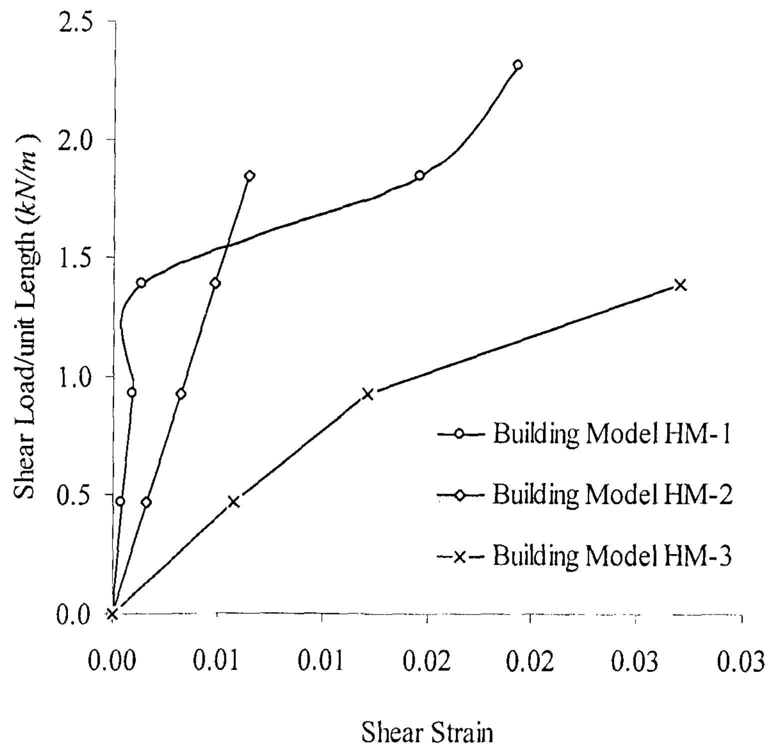


Fig. 4.21 Shear Load-Strain Curves for Different Building Models

Table 4.11 Maximum Lateral Load and Corresponding Deformation in Building Models

Type of models	Type of walls	Maximum average deformation of diagonal		Ultimate lateral load (*kN*)	Maximum deflection at top (*mm*)
		Elongation (*mm*)	Contraction (*mm*)		
HM-1 (Masonry Type A)	Wall with door Wall with window	0.840 0.595	0.995 0.855	2.5	15.85
HM-2 (Masonry Type B)	Wall with door Wall with window	0.950 5.075	0.325 2.720	2.0	8.52
HM-3 (Masonry Type D)	Wall with door Wall with window	1.695 10.650	0.425 2.620	1.5	22.15

4.4.3. Lateral Stiffness of Building Models

The lateral stiffness of walls of building models tested experimentally has been calculated analytically using Eqs. (4.11) to (4.14) depending upon the type of opening and the values are given in Table 4.12. The thickness of wall for different types of masonry is taken as the effective thickness of wall. The two shear walls of the building models are having different opening – one wall has a door and the other wall has a window, therefore, the lateral displacement of wall is due to the combined effect of both the walls. It is due to this reason that the analytical value of stiffness of wall is taken as the sum of the stiffness of wall with door and wall with window opening. It is observed from the table that the consideration of shear deformation in the calculation of lateral stiffness of wall reduces the lateral stiffness of different types of building models by 82.66%.

Table 4.12 Lateral Stiffness of Different Types of Brick Masonry Building Models

Type of masonry	Lateral stiffness (*kN/mm*)					
	Wall with door opening		Wall with window opening		Building Model	
	Without shear	With shear	Without shear	With shear	Without shear	With shear
A	145.5	23.2	145.8	27.3	291.3	50.5
B	170.3	27.2	170.6	31.9	340.9	59.1
D	186.7	29.8	187.1	35.0	373.8	64.8

4.5. DYNAMIC RESPONSE OF BUILDING MODELS

The identification mark inscribed on the building models are such that the first character is the masonry type and the second and third characters tell about the type of base ("R" stands for the rigid base and "BI" stands for the base isolation). The identification marks for different models are:

AR: Building model constructed with masonry Type A with rigid base
ABI: Building model constructed with masonry Type A with base isolation
BR: Building model constructed with masonry Type B with rigid base
BBI: Building model constructed with masonry Type B with base isolation
DR: Building model constructed with masonry Type D with rigid base
DBI: Building model constructed with masonry Type D with base isolation

The following nomenclature has been used for the identification of walls of building models:

W1: Wall having door (1.1 *m* long)
W2: Wall having window (1.1 *m* long)
W3: Wall on which displacement was recorded (1.0 *m* long)
W4: Wall opposite to the wall on which displacement was recorded (1.0 *m* long)

The direction of shaking was parallel to the walls: W1 and W2. The building models were tested at 140 *deg* eccentricity and different speed of motor, which was increased from 300 to 3000 *rpm* in steps of 300 *rpm*. The displacement, acceleration and crack pattern observed from experimental testing have been discussed in the subsequent sub-sections.

4.5.1. Mathematical Model for Building

A single-storey building model as shown in Fig. 4.22 represents the concept of single storey structure with base sliding. It is assumed that a layer of suitable material with known coefficient of friction is laid between the contact surface of plinth band of superstructure and the plinth band of substructure. The superstructure is allowed to slide freely after overcoming the friction. The sliding type of building is idealized as a discrete mass model with two degrees of freedom for computing the earthquake response as shown in Fig. 4.23. The spring action in the system is assumed to be provided by the shear walls, which resist shear force parallel to the direction of earthquake shock. Internal damping is represented by a dashpot that is in parallel with the spring. The mass of roof slab and one half of the wall is lumped at the top (= M_t) and the lower mass, M_b, is assumed to rest on a plane with dry friction damping.

Further assumptions made in the analysis of the sliding system are as follows:

- The coefficient of friction between the sliding surfaces is assumed to be constant throughout the motion of the system.
- The materials used for the building construction are linearly elastic within the limit of proportionality, thus the idealized spring is linear elastic. Its stiffness is computed by considering bending as well as shear deformations in the wall element.
- The sliding displacement can occur at the contact surface without overturning or tilting.

- The building is assumed to be subjected to only one horizontal component of ground motion at a time. The effect of vertical ground motion is not considered.

There are three different phases in the complete motion history of the base sliding system due to the frictional resistance at its base. The equations of motion are given in the following:

(i) Phase I

Initially, so long as the acceleration of the sliding system does not overcome the frictional resistance, bottom mass, M_b, moves with the base since there is no sliding and the system behaves as a single degree of freedom system. Therefore, the equation of motion is:

$$M_t \ddot{X}_t + C_s(\dot{Z}_t - \dot{Z}_b) + K_s(Z_t - Z_b) = 0 \qquad \ldots(4.15)$$

$$\ddot{Z}_t + 2\omega\xi(\dot{Z}_t - \dot{Z}_b) + \omega^2(Z_t - Z_b) = -\ddot{y}(t) \qquad \ldots(4.16)$$

where, C_s = coefficient of viscous damper

K_s = spring constant

M_t = mass lumped at the roof level

M_b = mass lumped at the base

$\omega = \sqrt{\dfrac{K_s}{M_t}}$ = natural circular frequency of the system

$\ddot{X}_t$, $\ddot{Z}_t$ = absolute and relative accelerations of the top mass respectively

$\ddot{y}(t)$ = ground acceleration at time t

Z_b, Z_t = lateral relative displacements of masses M_b and M_t respectively

$\dot{Z}_t$, $\dot{Z}_b$ = relative velocities of masses M_b and M_t respectively

$\xi = \dfrac{C_s}{2\omega M_t}$ = fraction of critical damping *(ii)* ***Phase II***

The sliding of bottom mass begins when the force which causes sliding overcomes the frictional resistance at the plinth level. The force to cause sliding S_f is given by:

$$S_t = C_s\left(\dot{Z}_t - \dot{Z}_b\right) + K_s\left(Z_t - Z_b\right) - M_b\ddot{X}_b \qquad \text{...(4.17)}$$

Sliding of mass occurs if

$$\left|S_f\right| \rangle \ \mu M_T g$$

where, g = acceleration due to gravity

M_T = $M_b + M_t$

μ = coefficient of friction

The system now acts as two degrees of freedom system for which the equation of motion can be written in simplified form as:

$$\ddot{Z}_b + 2\omega\xi\theta\left(\dot{Z}_t - \dot{Z}_b\right) + \omega^2\theta\left(Z_t - Z_b\right) + F = -\ddot{y}(t) \qquad \text{...(4.18)}$$

and $$\ddot{Z}_t + 2\omega\xi\left(\dot{Z}_t - \dot{Z}_b\right) + \omega^2\left(Z_t - Z_b\right) = -\ddot{y}(t) \qquad \text{...(4.19)}$$

where, $F = \mu g(1+\theta)sgn\left(\dot{Z}_b\right)$ ...(4.20)

$\ddot{X}_b$, $\ddot{Z}_b$ = absolute and relative accelerations of bottom mass, M_b

$\theta = \dfrac{M_t}{M_b}$ = mass ratio

$sgn\left(\dot{Z}_b\right)$ = +1, if $\dot{Z}_b$ is positive

= -1, if $\dot{Z}_b$ is negative

(iii) Phase III

At any time during the motion of the system, if

$$\left|S_f\right| \leq \mu M_T g \qquad \ldots(4.21)$$

then the sliding of the bottom mass is stopped but the top mass continues to vibrate and the system again converts to a single degree of freedom system as considered in Phase I. Throughout the history of ground shaking, the bottom mass of the system either stops or continues to slide according to the conditions enumerated earlier.

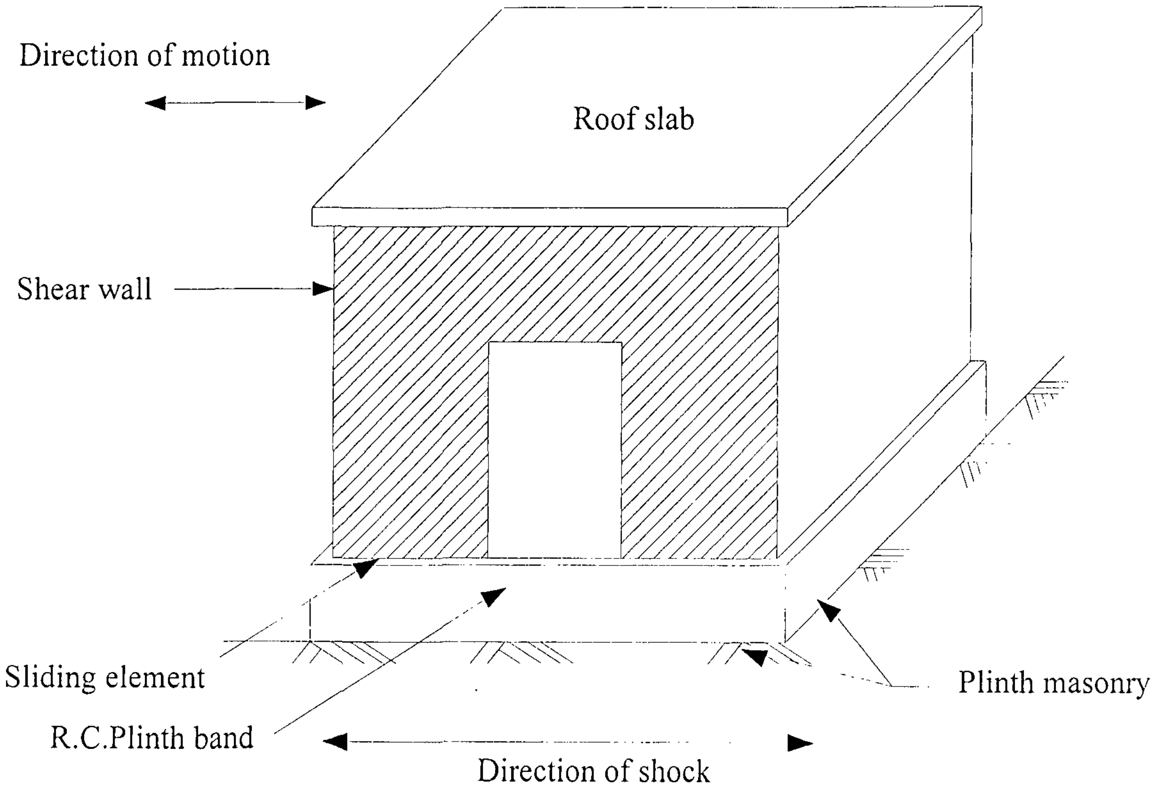


Fig. 4.22 Idealized Sliding Type Single Storey Structure

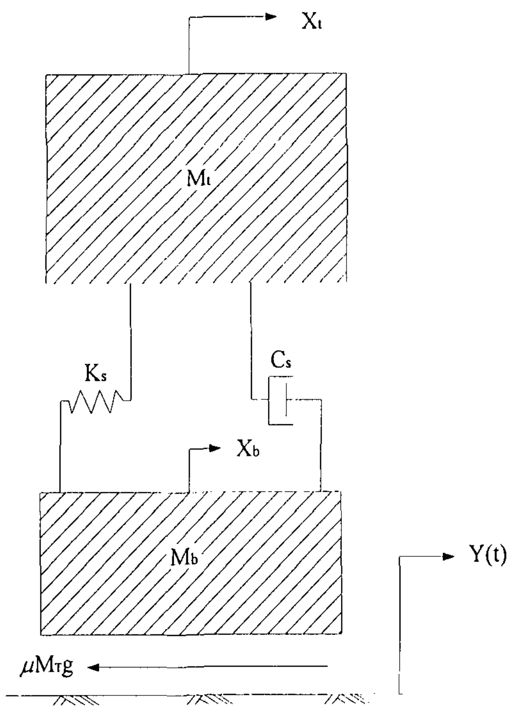


Fig. 4.23 Mathematical Model for Single Storey Structure with Sliding

4.5.2. Crack Pattern

(i) Building Model AR (Fig. 4.24)

Speed ≤ 1800 rpm

In this model, no crack was observed up to 1800 *rpm*.

Speed = 2100 rpm

Three horizontal cracks were observed in the wall W1. One crack was two courses below the roof level, running throughout the length of the wall. The other two cracks were located at the top and bottom level of the door on the same side of the door. In the wall W2, three horizontal cracks were observed at a speed of 2100 *rpm*. Out of these three cracks, one crack was located two courses below the roof level covering the whole length of the wall, second crack was at the lintel level of the window and third crack was at the bottom level of the window only on one side of the window. On the wall W3, two horizontal cracks were observed two courses below the roof level running throughout the length of the wall. One horizontal crack was also observed five courses above the base of the wall only on half length of the wall. On the wall W4, one horizontal crack was observed two courses below the roof level running throughout the length of the wall. Some small horizontal cracks were also observed at the base of the wall.

Speed = 2700 to 3000 rpm

At a speed of 2700 *rpm*, one vertical crack below the window passing from the mid length of the window was observed and some small horizontal cracks were also observed at the base of wall W2. With further increase in the speed no additional cracks were observed on any of the walls of building model. Only the cracks, already formed, widened in their width.

(ii) Building Model ABI (Fig. 4.25)

Speed ≤ 2400 rpm

In this model no crack was observed up to a speed of 2400 *rpm*.

Speed = 2700 rpm

Three horizontal cracks and one vertical crack were observed on the wall W1. One horizontal crack was located just below the roof starting from the corner and running upto two-brick length; second horizontal crack was one course below the roof level starting from the corner and the crack became vertical at the door level and came down upto the top corner of the door. The third horizontal crack was just below the second horizontal crack only for one brick length near the corner. In the wall W2, three horizontal cracks were observed. The first crack was located just below the roof level starting from the corner and running upto the middle of the wall. The other two cracks were only one brick length long. One crack near the opposite corner, one course below the roof level and the other crack was near the top corner of the window and below the previous small crack.

Speed = 3000 rpm

In the wall W4, three horizontal cracks were observed. The first crack is just below the roof level covering the whole length of the wall and the other two cracks are only one brick length long located near the opposite corners and one course below the first crack. No crack was observed on the wall W3.

(iii) Building Model BR (Fig. 4.26)

Speed ≤ 2100 rpm

In this model no crack was observed up to a speed of 2100 *rpm*.

Speed = 2400 rpm

Cracks appeared on all the walls except the wall W2. In the wall W1, one horizontal crack was observed at one of the bottom corner which was one brick long just one course above the base of the wall then it became vertical passing through the vertical mortar joint for one brick thickness reaching to the bottom of the wall then again the crack became horizontal and extended upto the bottom of the door.

Speed = 2700 rpm

New cracks were not formed, only cracks already formed, widened.

Speed = 3000 rpm

In the wall W4, two bricks laid on edge in the second course from bottom of the wall near one corner got detached from the wall. One brick got detached from the wall W3 at 2400 *rpm* at the corner from the bottom course, whereas one more brick came out at 3000 *rpm* from the opposite corner from the same course. The bricks that get detached were the bricks on edge and these bricks were located near the corners of the walls.

(iv) Building Model BBI (Fig. 4.27)

In this model, no crack was observed in different walls upto 2700 *rpm* but at 3000 *rpm* sliding of the slab took place due to the development of horizontal cracks throughout the perimeter of the model at the roof level. No other crack was observed in model.

(v) Building Model DR (Fig. 4.28)

Speed ≤ 2100 rpm

In this model, no crack was observed in the walls upto 2100 *rpm*.

Speed = 2400 rpm

In the wall W1, one horizontal crack developed at two courses above the base of the wall starting from one corner and running upto the door then from the opposite end of the door for one brick length which remained horizontal and thereafter it became vertical through the vertical mortar joint and again it became horizontal reaching upto the other corner of the wall. In the wall W2, one horizontal crack located one course below the roof level and running along the length of the wall was observed.

Speed = 2700 rpm

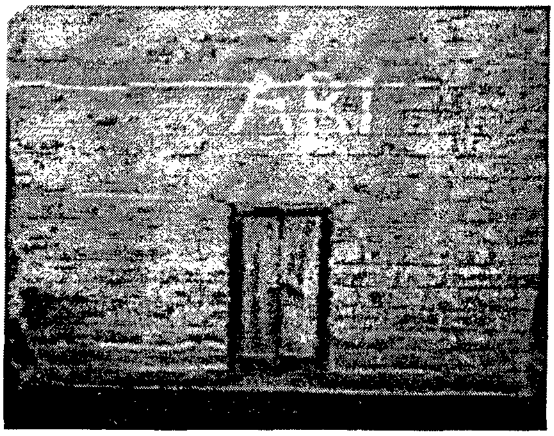
In the wall W1, one diagonal crack appeared starting from one of the corners of the door and covering two courses. Bricks got detached from the two corners, one and two courses above the base of the wall. In the wall W3, two horizontal cracks were observed, one crack was running along the length of the wall and located three courses from the base and the other crack was just one course below the roof level and started from one corner running only half length of the wall.

Speed = 3000 rpm

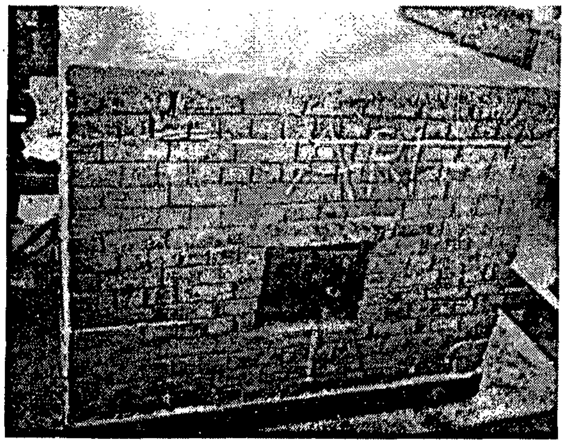
The brick on edge got detached because of the lateral inertial force on the brick being greater than the sum of the bond strength of vertical face of brick with mortar and the shear strength of horizontal mortar bed.

(vi) Building Model DBI (Fig. 4.29)

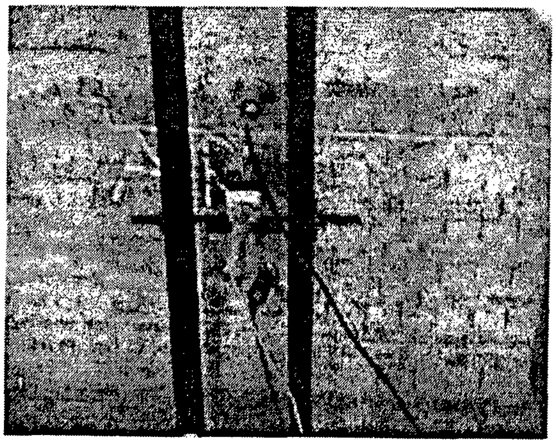
In this model no crack was observed upto 2400 *rpm* but at 2700 *rpm* few minor horizontal cracks at the roof level appeared in the walls of the model. At this speed one small diagonal crack was also observed at one corner of the door covering two courses in the wall W1. At the maximum speed of 3000 *rpm*, a number of bricks got detached from the top course of all the walls.



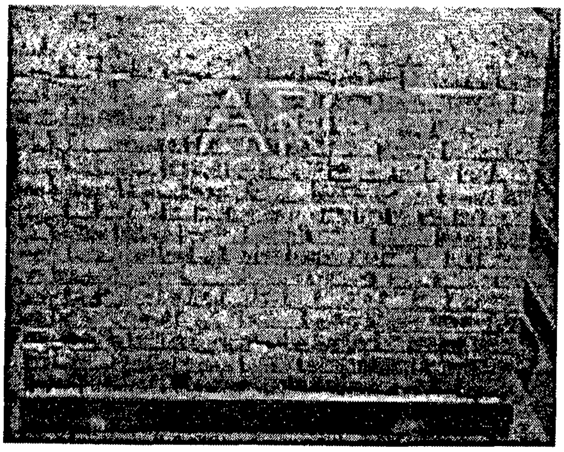
(a) Wall – W1: Wall having Door



(b) Wall – W2: Wall having window

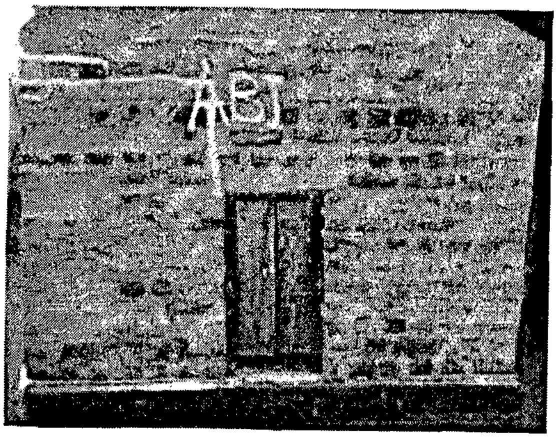


(c) Wall – W3

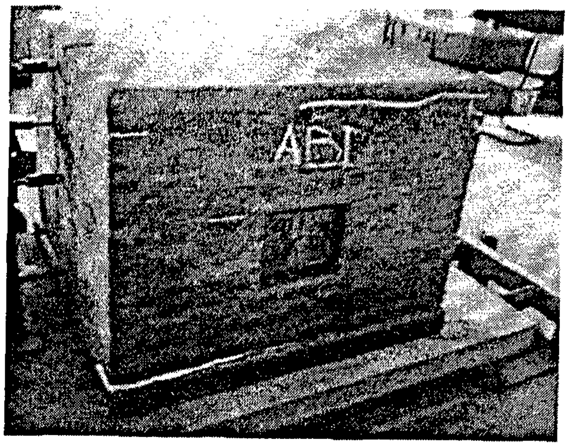


(d) Wall – W4

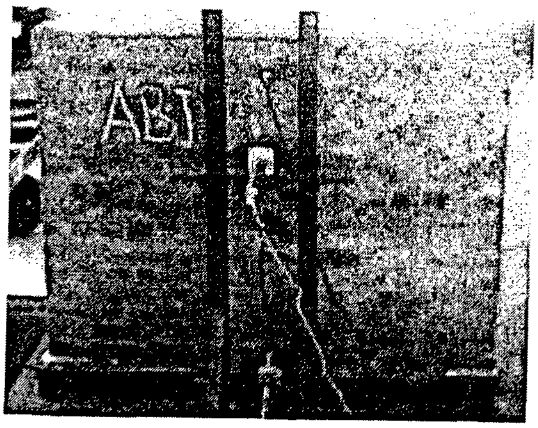
Fig. 4.24 Crack Pattern in Building Model AR under Shake Table Testing



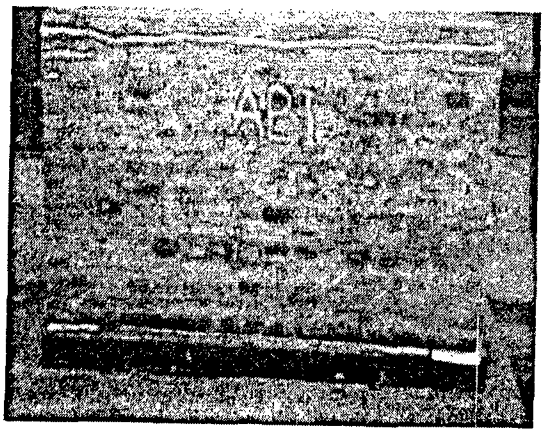
(a) Wall – W1: Wall having Door



(b) Wall – W2: Wall having window

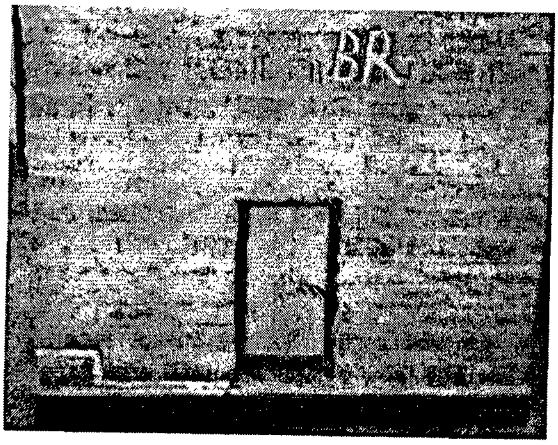


(c) Wall – W3

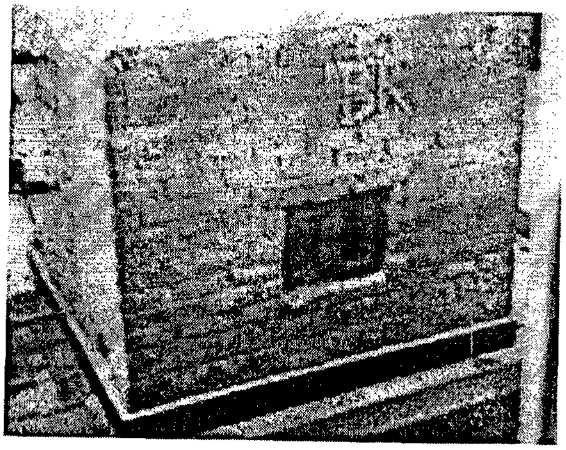


(d) Wall – W4

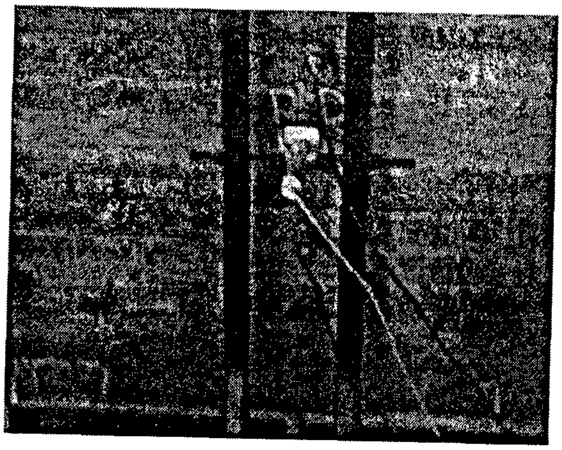
Fig. 4.25 Crack Pattern in Building Model ABI under Shake Table Testing



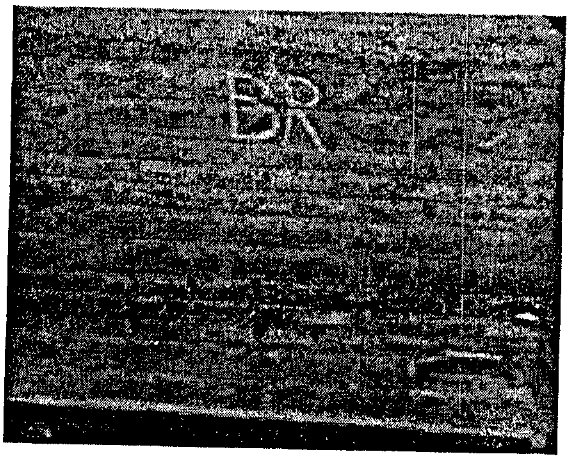
(a) Wall – W1: Wall having Door



(b) Wall – W2: Wall having window



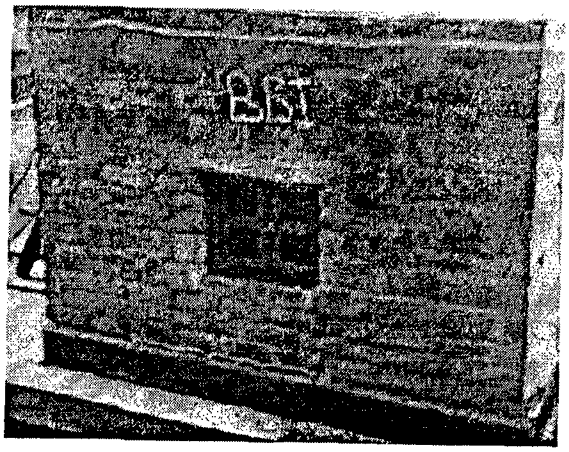
(c) Wall – W3



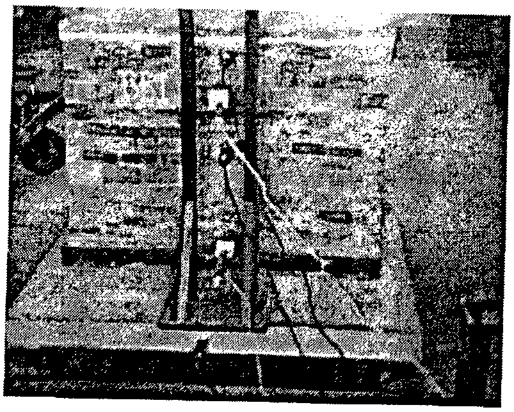
(d) Wall – W4

Fig. 4.26 Crack Pattern in Building Model BR under Shake Table Testing

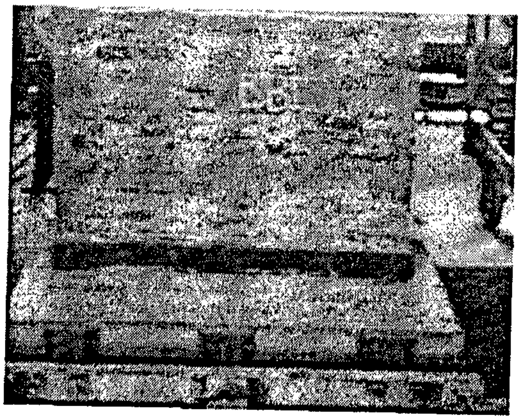
(a) Wall – W1: Wall having Door



(b) Wall – W2: Wall having window

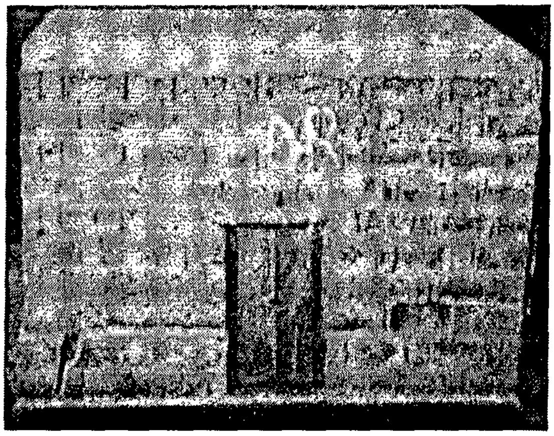


(c) Wall – W3

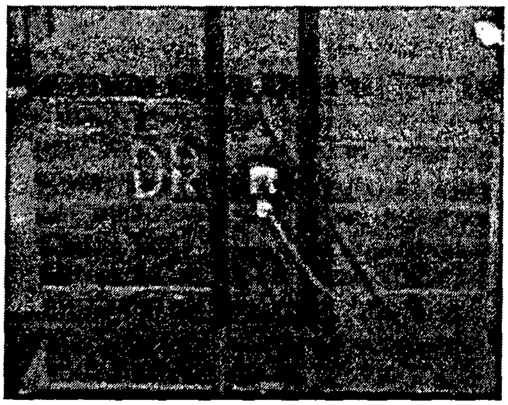


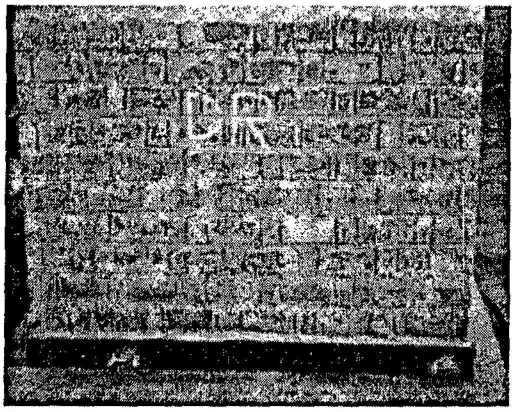
(d) Wall – W4

Fig. 4.27 Crack Pattern in Building Model BBI under Shake Table Testing



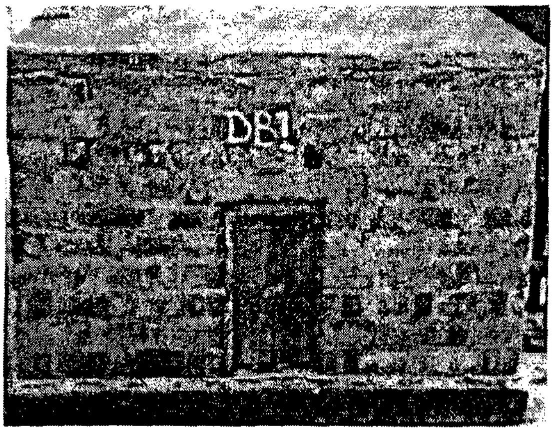
(a) Wall – W1: Wall having Door (b) Wall – W2: Wall having window



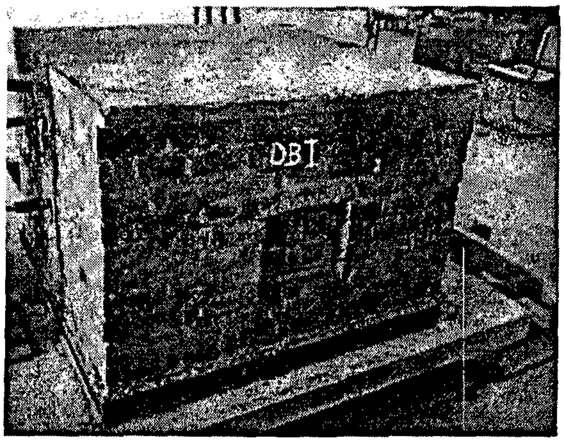


(c) Wall – W3 (d) Wall – W4

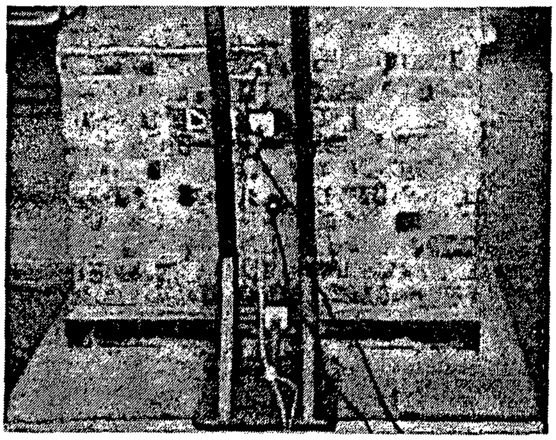
Fig. 4.28 Crack Pattern in Building Model DR under Shake Table Testing



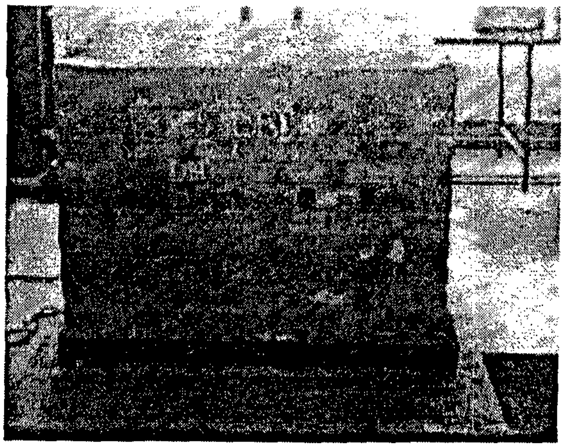
(a) Wall – W1: Wall having Door



(b) Wall – W2: Wall having window



(c) Wall – W3



(d) Wall – W4

Fig. 4.29 Crack Pattern in Building Model DBI under Shake Table Testing

4.5.3. Acceleration Response

The brick masonry building models were subjected to gradually increasing shake table accelerations. The variation of acceleration at top of building models with shake table accelerations (base accelerations) have been plotted in Figs. 4.30 to 4.32. The theoretical values of acceleration are also plotted in these figures. From the figures it is seen that the theoretical vales are in good agreement with the experimental values with the percentage error for all the cases less than 10%. The variation is almost linear for all types of building models with and without base isolation.

With the increase in shake table acceleration, the acceleration at top of all building models also increases. This is due to increase of dynamic force to which models are subjected. The experimental results indicate that the acceleration at the top of base isolated building models (ABI, BBI and DBI) is less than the building models without base isolation (AR, BR, and DR). This shows that the dynamic force transmitted to the top of base isolated building models is considerably reduced due to the provision of base isolation.

The accelerations recorded by the accelerometers at top of different building models and at the shake table are given in Tables 4.13 to 4.15 and discussed in the following. In these tables, only those accelerations of shake table are reported at which the acceleration at the top of different building models started decreasing either due to all-round horizontal crack formation at the plinth level in building models AR, BR and DR, due to which these models behaved like sliding type models or due to sliding of base isolated building models ABI, BBI and DBI.

<u>Masonry Type – A:</u>

- The percentage reduction in the acceleration at top of model AR with respect to shake table acceleration varies from 20% to 31% whereas for model ABI this reduction is from 37% to 51%.

- In building model ABI sliding started at an acceleration of 1.27 *g*, the sliding recorded at this acceleration was 19 *mm* which is 8.33% of the thickness of wall. With further increase in shake table acceleration, the magnitude of sliding had increased. The maximum sliding of 51 *mm* (22.37% of the thickness of wall) was recorded at shake table acceleration of 2.95 g.

<u>Masonry Type – B:</u>

- The percentage reduction in the acceleration at top of model BR with respect to shake table acceleration varies from 23% to 36% whereas for model BBI this reduction is from 40% to 48%.
- In building model BBI, sliding started at an acceleration of 1.24 *g*, the sliding recorded at this acceleration was 23 *mm* which is 12.11% of the thickness of wall. With further increase in shake table acceleration, the magnitude of sliding had increased. The maximum sliding of 55 *mm* (28.95% of the thickness of wall) was recorded at shake table acceleration of 2.98 *g*.

<u>Masonry Type – D:</u>

- The percentage reduction in the acceleration at top of model DR with respect to shake table acceleration varies from 11% to 32% whereas for model DBI this reduction is from 37% to 43%.
- In building model DBI sliding started at an acceleration of 1.35 *g*, the sliding recorded at this acceleration was 30 *mm* which is 13.16% of the thickness of wall. With further increase in shake table acceleration, the magnitude of sliding had increased. The maximum sliding of 58 *mm* (25.44% of the thickness of wall) was recorded at shake table acceleration of 2.94 *g*.

In view of the above data, obtained for different building models by shake table testing, following conclusions may be drawn:

- In the building models ABI, BBI and DBI higher reduction in the acceleration was observed as compared to building models AR, BR and DR, this is due to the provision of base isolation in models ABI, BBI and DBI. This observation indicated that the base isolation resulting from sliding had an appreciable influence and that the input energy at the base was dissipated to some extent in the sliding of the model.
- Type D building models had suffered maximum sliding as compared to Type B and A building models because Type D building model is lightest among the three type of building models.

Table 4.13 Acceleration and Sliding of Building Models of Masonry Type A

Model type	Acceleration (*g*)		Reduction in top acceleration (%)	Sliding (*mm*)
	Shake table	At top of model		
AR	1.27	1.01	20	-
	1.63	1.30	20	-
	1.88	1.45	23	-
	2.34	1.65	29	-
	2.59	1.87	28	-
	2.85	1.96	31	-
ABI	1.27	0.69	37	19
	1.43	0.78	46	26
	1.88	0.99	45	37
	2.14	1.21	47	42
	2.49	1.01	48	48
	2.95	1.42	51	51

Table 4.14 Acceleration and Sliding of Building Models of Masonry Type B

Model type	Acceleration (*g*)		Reduction in top acceleration (%)	Sliding (*mm*)
	Shake table	At Top of model		
BR	1.20	0.93	23	-
	1.58	1.13	28	-
	2.03	1.37	33	-
	2.54	1.63	36	-
	2.83	1.83	35	-
BBI	1.24	0.73	41	23
	1.49	0.86	42	35
	1.93	1.16	40	42
	2.27	1.25	45	46
	2.98	1.54	48	55

Table 4.15 Acceleration and Sliding of Building Models of Masonry Type D

Model type	Acceleration (*g*)		Reduction in top acceleration (%)	Sliding (*mm*)
	Shake table	At Top of model		
DR	1.32	1.17	11	-
	1.55	1.23	21	-
	1.93	1.45	25	-
	2.60	1.77	32	-
	2.79	1.95	30	-
DBI	1.35	0.85	37	30
	1.94	1.10	43	38
	2.02	1.17	42	49
	2.48	1.45	42	54
	2.94	1.75	40	58

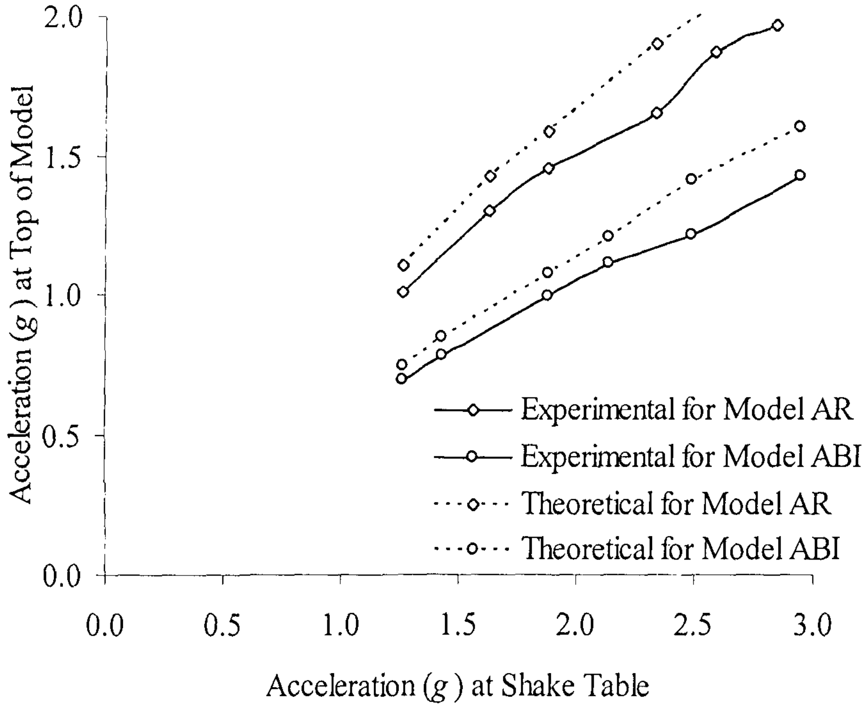


Fig. 4.30 Base Acceleration Vs Acceleration at the Top of Type A Building Models

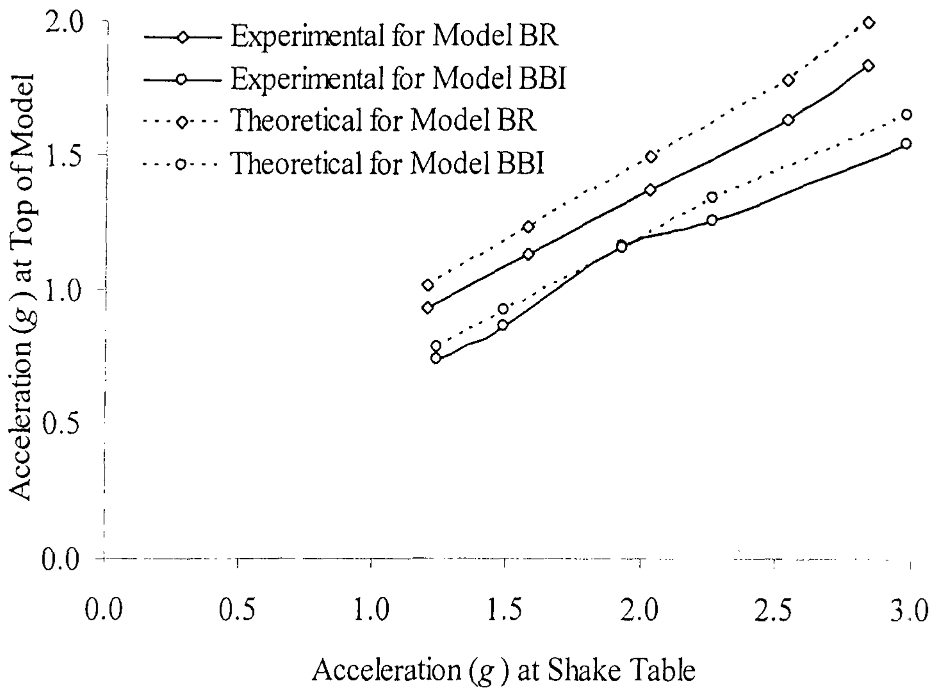


Fig. 4.31 Base Acceleration Vs Acceleration at the Top of Type B Building Models

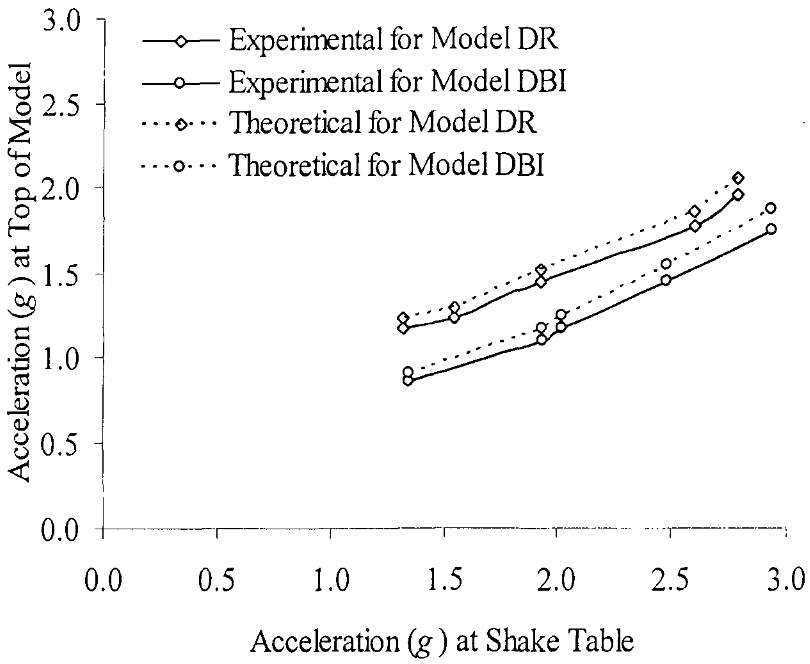


Fig. 4.32 Base Acceleration Vs Acceleration at the Top of Type D Building Models

4.5.4. Displacement Response

The displacement recorded by two LVDTs, one each at top and base of a short wall (wall-W3) of different building models, are given in Table 4.16 and discussed in the following:

<u>Masonry Type – A:</u>

- The maximum displacement recorded for model AR at the top is 23.96% more than the displacement recorded at the base, whereas, for building model ABI this increase is 23.04%. The displacement recoded at the top and the base for building model ABI is 18.56% and 19.45% more than the building model AR, respectively.

Masonry Type – B:

- The maximum displacement recorded for model BR at the top is 19.12% more than the displacement recorded at the base, whereas, for building model BBI this increase is. 15.22%. The displacement recoded at the top and the base for building model ABI is 6.87% and 10.48% more than the building model BR, respectively.

Masonry Type – D:

- The maximum displacement recorded for model DR at the top is 14.05% more than the displacement recorded at base, whereas, for building model DBI this increase is. 17.74%. The displacement recoded at the top and the base for building model DBI is 20.77% and 17.00% more than the building model DR, respectively.

Table 4.16 Maximum Displacement of Different Building Models

S. No.	Building model	Maximum displacement (*mm*) for building models	
		At the top	At the base
1	AR	10.45	8.43
2	ABI	12.39	10.07
3	BR	12.96	10.88
4	BBI	13.85	12.02
5	DR	13.96	12.24
6	DBI	16.86	14.32

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS

5.1. GENERAL

The conclusions derived from the present study are based on the limited experimental and analytical studies of conventional and three types of low cost brick masonry presented in previous Chapters. The seismic response of masonry buildings has been assessed from the shake table testing of single room for unidirectional sinusoidal horizontal motion. The conclusions regarding the mechanical properties of brick masonry, lateral load carrying capacity as well as shear stiffness of wall panels and the response of building models under static as well as dynamic loads are presented in subsequent sections.

Four types of brick masonry viz. Type A, B, C and D have been used in the present study. The brick masonry Type A and B are solid, whereas Type C and D are having inside cavity. In brick masonry Type A, bricks in every course are flat, in Type B one brick is flat and one on-edge in every course. In masonry Type C, there are alternate header and stretcher courses, header course is of flat bricks and stretcher course is of on-edge bricks. In masonry Type D, bricks in every course are on-edge with alternate header and stretcher bricks.

5.2. MECHANICAL PROPERTIES

The conclusions drawn from the testing of brick masonry prisms of the four types of brick masonry are presented in subsequent subsections.

5.2.1. Mode of Cracking

The conclusions drawn from the crack pattern observed in the testing of masonry prisms are:

i) In most of the brick masonry prisms, cracks developed along the vertical mortar joints. This may be due to the following reasons:

- Even if care is taken in laying of bricks, all bricks will not be evenly supported on the mortar bed due to which bricks are subjected to flexural and shear stresses along the vertical mortar joints. This may be responsible for the cracking along the vertical joints and these additional stresses will reduce the strength of prisms.

- Insufficient filling of mortar joints, varying thickness of bricks and joints give rise not only to the flexural stresses but also result in uneven distribution of external load. It is due to these reasons that the stress concentration develops in the bricks, which may be considerably larger than the nominal average stress which may also be the cause of failure.

ii) The prisms of masonry Type A failed by the development of vertical cracks along the vertical mortar joint which is due to the following reasons:

- When masonry prism is loaded, the brick and mortar expand laterally because of the Poisson's effect. Since mortar expands more than the bricks, the bricks are subjected to lateral tension which is more than what they experience under uniaxial compression. Therefore the bricks fail in lateral tension.

- Another reason for the development of vertical crack along vertical mortar joint is that the lateral tensile strength of the vertical mortar joint is less than the tensile strength of bricks.

iii) The prisms of masonry Type B failed by the development of vertical cracks at mid length of bricks. In this type of brick masonry cracking is not along the vertical mortar joint because all of the vertical mortar joints are not in same vertical line. The cracking of bricks at mid length is due to lateral tension getting developed in the bricks because of the lateral expansion, due to the Poisson's effect.

iv) In the prisms of masonry Type C, the failure is by the development of vertical cracks in the bricks on the bed. This is because of the lateral tension getting developed in these bricks.

v) In the masonry Type D, vertical cracks have developed in the header bricks which are due to the development of lateral tension because of the Poisson's effect. Vertical cracks have also developed along the vertical mortar joint which is due to the development of lateral tension in bricks in the transverse direction.

vi) All the masonry prisms tested in shear failed by sliding along the horizontal mortar bed which is the weakest plane in shear.

5.2.2. Compressive and Shear Strength

Major conclusions drawn regarding the compressive and shear strength of brick masonry are:

i) Mechanical properties of conventional and three types of low cost brick masonry have been determined and mathematical models have been developed for their estimation. The shear strength of brick masonry is proportional to the square root of the compressive strength of brick masonry.

ii) Crushing strain of masonry Type A is maximum. Out of the three low cost brick masonry options, crushing strain is maximum for masonry Type B and minimum for masonry Type C, which shows that masonry Type B is more ductile as compared to the other two. However, the crushing strain of masonry Type D is only 9 percent less than the masonry Type B.

iii) The compressive strength of masonry Type D is maximum. This is due to the fact that number of mortar joints for the same height in masonry type D is less than other masonry types, due to laying of all the courses of bricks on edge. But the vertical load carrying capacity of masonry Type A is more as compared to the masonry Type B, C, and D. It is because the load carrying capacity is based on the net area of the cross section which is maximum in the masonry Type A.

iv) The compressive strength of masonry Type A, B and C is 87, 85 and 60 percent of masonry Type D respectively. The vertical load carrying capacity of masonry Type B, C and D is 81, 60 and 88 percent of masonry Type A respectively.

v) The compressive strength of brick masonry built with full size bricks is found to be 40% of the brick masonry built with half size bricks for all types of masonry.

vi) The shear strength as well as the shear load carrying capacity is maximum for masonry Type A. The shear strength of masonry Type B, C and D is 80, 48 and 79 percent of masonry Type A respectively. The shear load carrying capacity of masonry Type B, C and D is 62, 42 and 60 percent of masonry Type A.

vii) The shear strength of brick masonry built with full size bricks is found to be 63% of the brick masonry built with half size bricks for all types of masonry.

The low cost brick masonry Type C was not considered for the further study because of its lowest value of compressive and shear strength.

5.3. BEHAVIOUR OF WALL PANELS UNDER STATIC LOAD

On the basis of test results obtained by testing brick masonry wall panels and by finite element analysis of wall panel, the following conclusions have been drawn:

i) The ultimate lateral load carrying capacity for wall panel of brick masonry Type B and D is 29 and 38 percent less than that of masonry Type A respectively.

ii) The shear strength of wall panel for brick masonry Type B and D is 13 and 6.5 percent less than that of masonry Type A respectively.

iii) The peak lateral deflection of wall panel of masonry Type B and D is 77 and 82 percent higher than the peak deflection of wall panel of masonry Type A respectively.

iv) The peak lateral deflection of wall panel of brick masonry Type D is 3 percent higher than wall panel of brick masonry Type B.

v) The shear stresses and maximum absolute stresses increase as the thickness of mortar bed is increased in the three type of brick masonry. It holds good for all types of brick masonry considered in the study.

vi) The effect of inclusion of shear deformation in the calculation of lateral stiffness of different types of masonry wall without any structural opening is 50 percent.

vii) If a ventilator is provided above a door opening (both placed at mid length of wall), the effect of change in the position of ventilator on the lateral stiffness of wall is almost negligible.

viii) The presence of opening such as a window of size 1.8×1.95 *m* in a wall panel of 3×3 *m* in size reduces the lateral stiffness of wall by 58 percent.

ix) The wall panels of masonry Type A and B failed due to sliding at the base along the horizontal mortar bed which is because of the masonry being stronger along the diagonals. Whereas the wall panel of masonry Type D failed in diagonal tension by the development of cracks along the diagonal passing through horizontal and vertical mortar joints because this type of masonry is relatively weaker in tension along its diagonals. The reduction in tensile strength along the diagonal is because of the vertical mortar joint being relatively weaker because the height of vertical mortar joint in masonry Type D is more as compared to masonry Type A. The cracks in the wall panels of all types of brick masonry got developed only at the failure load and there was no any visible crack at lesser magnitude of load.

This can be concluded that the masonry Type B is stronger in carrying lateral load as compared to masonry Type D but the difference is only 12 percent. Whereas, the ultimate shear strain is maximum for masonry Type D which shows that this type of masonry is more ductile under lateral load as compared to other types of masonry. The presence of opening as well as the consideration of shear deformation considerably reduces the stiffness of wall.

5.4. BEHAVIOUR OF BUILDING MODELS UNDER STATIC LOAD

From the study of load deflection on building models HM-1 (Masonry Type A), HM-2 (Masonry Type B) and HM-3 (Masonry Type D) under static lateral load, the following conclusions are drawn:

i) The lateral load carrying capacity of building model HM-1 is maximum and for building models HM-2 and HM-3 it is 20 and 40 percent less than building model HM-1 respectively. Whereas, the peak lateral deflection of building model HM-3 is maximum; for building models HM-1 and HM-2 it is 28 and 61 percent less than HM-3 respectively.

ii) Out of the two low cost brick masonry building models, building model HM-2 is found to be 33 percent stronger than HM-3 in carrying the lateral load and the peak lateral deflection of the top of building model HM-3 is 160 percent more than building model HM-2.

iii) The consideration of shear deformation in the calculation of lateral stiffness of shear walls of the building models reduces the lateral stiffness of shear walls of building models by 82 percent.

iv) The cracks developed in the models are mainly diagonal, whereas, some of the cracks are horizontal and close to the slab. All the cracks observed in different building models are along the mortar joints.

v) The observed pattern of diagonal cracks in the shear walls are due to the combined effect of elongation of far-diagonal, contraction of near-diagonal, fixity at the base and brittleness of the masonry. Whereas, the cracks near the edges may be due to some invisible eccentricity and stress concentration. The pattern of dominant horizontal cracks in the cross walls are due to combined effect of lateral deflection of building in the direction of load and brittleness of brick masonry which reduces the shear strength of brick masonry. The different types of crack patterns observed on different faces of building models provide strengthening criteria of such buildings against destructive forces that are likely to act on buildings during service condition.

vi) The crack patterns observed in wall panels tested independently under lateral load and the walls of building models are quite different. This is because of the box action and the presence of openings in the walls of the building models.

Among the two types of low cost brick masonry used in the construction of building models, brick masonry Type B has more ultimate lateral strength and moderate ductility where as the brick masonry Type D has greater ductility and the lateral strength is 25 percent lower than that of masonry Type B.

5.5. BEHAVIOUR OF BUILDING MODELS UNDER DYNAMIC LOAD

Based on the shake table testing (unidirectional sinusoidal horizontal motion) of different types of building models with and without base isolation, the following conclusions have been drawn:

i) In the building models ABI, BBI and DBI, higher reduction in the acceleration was observed as compared to building models AR, BR and DR, this is due to the provision of base isolation in models ABI, BBI and DBI.

ii) The building models of masonry Type D undergone maximum sliding as compared to building models of masonry Type A and B, because the building model of masonry Type D is lightest among the three type of building models.

iii) The peak deflection is maximum for building models of masonry Type D with and without base isolation. This shows that building model of masonry Type D is more ductile as compared to building models of masonry Type A and B.

iv) The vertical mortar joints are more important in the low cost brick masonry construction because of the use of bricks on edge in all types of low cost masonry. The vertical mortar joints are especially more important for brick masonry Type B where mixed arrangement of bricks is there.

This study confirmed that the performance of building models with base isolation is much better as compared to building models without base isolation. Thus the sliding arrangement shows great promise for adoption in actual building construction as a measure of earthquake safety.

On the basis of the mechanical properties, behaviour of wall panels and the performance of building models under static as well as dynamic loads, the masonry Type D is found to be better than masonry Type B. Though the saving in the quantity of bricks and mortar as well as compressive strength is maximum in masonry Type D, but the shear strength is slightly less than that of the masonry Type B. The introduction of base isolation by providing Teflon sheet at the plinth level of the building improves its performance under earthquake forces.

5.6. SUGGESTIONS FOR FURTHER RESEARCH

It is suggested that prototype walls and buildings should be tested for studying their performance under static as well as seismic forces which will give better idea about their performance. The present study is limited only to the unidirectional horizontal motion of shake table testing. It is suggested that the buildings should be tested for the bi-directional horizontal and vertical motion of shake table. The effect of torsional component of ground motion should also be studied. In the present study, a building model with fixed aspect ratio and fixed location of openings has been studied, it is suggested that buildings should be tested by considering their variation. The effect of the thickness of mortar joint in the low cost masonry should also be studied.

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